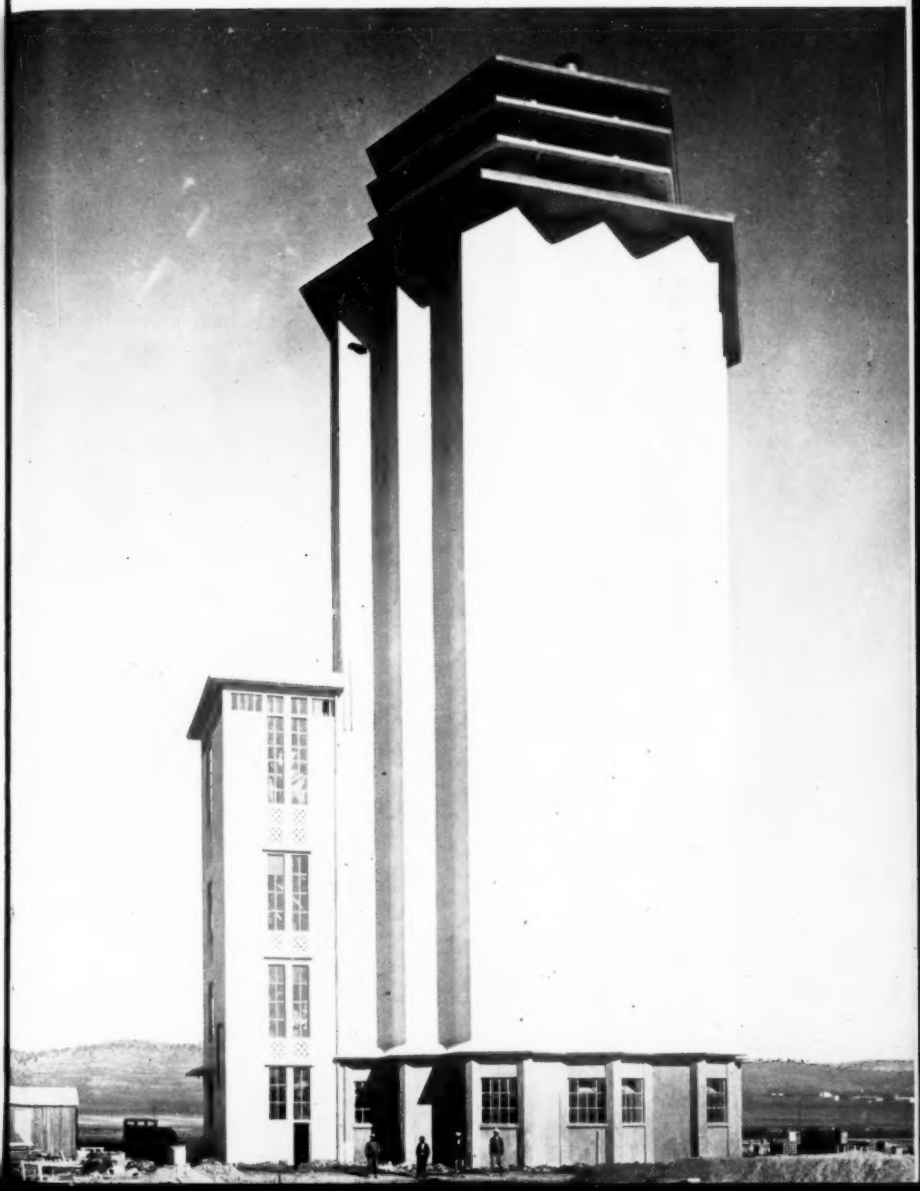
ACI News Letter

Vol. 21 No. 9 JOURNAL of the AMERICAN CONCRETE INSTITUTE May 1950



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News Letter Contents

May 1950

On the Cover—A hexagonal concrete grain elevator, Fez, French Morocco, described by G. L. Blanchard in Letters from Readers, p. 749.

Foundation laid for regional meeting..... 3

Positions and Projects—ACI Members..... 4

Honor Roll.....16

Who's Who

Committee 208.....17

Raymond C. Reese.....17

Arsham Amirikian.....17

George W. Washa and Paul G. Fluck.....17

Robert F. Blanks.....18

Miles N. Clair.....18

Curzon Dobell.....19

Thomas F. Gilbane.....19

C. D. Williams and F. Bromilow.....20

Errata.....20

New Members.....21

Tools, Materials, Services..24

Synopses.....25

Discussion.....31

ACI Publications.....33

List of advertisers.....34

Foundation laid for Washington regional meeting, October 24-25

Preparations got under way recently for the regional meeting to be held in Washington, D. C., October 24-25. ACI Members in and around Boston remember well the New England Regional Meeting, November, 1949, when about 200 registered; those Members around Birmingham, Ala., also recall the top-notch program at the regional meeting there in 1947. The Washington committee has promised a program equally interesting for the two-day affair in the capitol city.

E. W. Bauman, Managing Director, National Slag Assn., was named general chairman of the event. The committee includes Frank B. Brown, Managing Director, Wire Reinforcement Institute, registration; A. N. Carter, Manager, Highway Division, Associated General Contractors, publicity; H. F. Clemmer, Engineer of Materials, District of Columbia, entertainment; J. E. Dunn, District Engineer, Portland Cement Assn., inspection trips; A. T. Goldbeck, Engineering Director, National Crushed Stone Assn., program; S. S. Steinberg, Dean, College of Engineering, University of Maryland, educational contacts; and ACI President Frank H. Jackson, Bureau of Public Roads, as an ex officio member.

Plans for the regional meeting were discussed at a luncheon April 20 in Washington with 20 in attendance. The group included Mr. Bauman; Delmar E. Bloem (proxy for Stanton Walker), National Sand and Gravel Assn.; Mr. Brown; Mr. Clemmer; Roy W. Crum, Highway Research Board; Mr. Dunn; John R. Dwyer, National Bureau of Standards; Mr. Goldbeck; Y. K. Howat, Howat Concrete Co., Inc. (proxy for P. Y. K. Howat representing Engineers Club of Washington); Capt. A. D. Hunter, Bureau of Yards and Docks, Navy Dept. (proxy for Rear Admiral J. F. Jelley); Herbert Insley, National Bureau of Standards; Mr. Jackson; Harry J. Kirk, A. G. C. (proxy for Mr. Carter); Douglas E. Parsons, National Bureau of Standards; Perry H. Petersen, Housing and Home Finance Agency; Chester L. Post, Public Buildings Administration; Byram W. Steele, Office Chief of Engineers, Dept. of the Army; Dean Steinberg; L. W. Teller, Bureau of Public Roads; and Harvey Whipple, Secretary-Treasurer, American Concrete Institute.

Positions and Projects — ACI Members

Committee 115 plans new compilation

Committee 115, Research, has already begun work on next year's compilation of research projects. The annual compilation released recently listed 315 different projects being conducted by 52 organizations.

Since the committee is anxious to have complete coverage of all research projects in its next compilation, all persons having suggestions or knowing of omissions in this year's report are urged to initiate correspondence at once.

No information will be given publicity without the permission of the correspondent. Address your suggestions and inquiries to:

Prof. George W. Washa
Secretary ACI Committee 115
Education-Engineering Bldg.
University of Wisconsin
Madison 6, Wis.

Single copies of the compilation distributed at the 1950 convention are available to ACI members as long as the supply lasts. Address your request to American Concrete Institute, 18263 W. McNichols Rd., Detroit 19, Mich.

Research committee adds members

Committee 115, Research, announced the addition of 11 men to the committee and several replacements. Fred Hubbard replaced Harry J. Love as representative of the National Slag Assn. and S. N. Pearman replaced J. M. Martin as the representative of the South Carolina State Highway Dept.

Other new members include R. E. Copeland, National Concrete Masonry Assn.; Bengt Friberg, Granite City Steel Co.; B. G. Johnston, Lehigh University; George L. Kalousek, University of Toledo; Thomas B. Kennedy, U. S. Waterways Experiment Station; Robert L. Lewis, New York University; Walter J. McCoy, Lehigh Portland Cement Co.; R. E. Mills, Purdue University; R. H. Sherlock, University of Michigan; T. W. Thomas, University of Minnesota; and Kenneth B. Woods, Joint Highway Research Project, Purdue University.

Siess appointed to Committee 208

C. P. Siess, research assistant professor of civil engineering at the University of Illinois, Urbana, Ill., was recently appointed to Committee 208, Bond Stress.

Professor Siess is chairman of Committee 215, Fatigue of Concrete, and a member of Committee 321, Design of Reinforced Concrete Slabs, and Committee 115, Research.

He has had several papers published in the ACI JOURNAL and was awarded, with Nathan M. Newmark, the Wason Medal for "the most meritorious paper" for "Rational Analysis and Design of Two-Way Concrete Slabs," ACI JOURNAL, December, 1948.

Committee 314 adds members

Committee 314, Rigid Frame Bridges, has added four names to its roster: T. Y. Lin, Civil Engineering Dept., University of California, Berkeley, Calif.; James P. Michalos, Civil Engineering Dept., Iowa State College, Ames, Ia.; A. L. Parme, Portland Cement Assn., Chicago, Ill.; and Frederick A. Reickert, Dept. of Civil Engineering, University of Illinois, Urbana, Ill.

Archibald named to Committee 315

Raymond Archibald, Bureau of Public Roads, Washington, D. C., and chairman of the American Assn. of State Highway Officials Committee on Bridges and Structures, has been appointed to membership on ACI Committee 315, Detailing Reinforced Concrete Structures.

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Kerekes joins Committee 318

Frank Kerekes, Iowa State College, Ames, Ia., was appointed recently to Committee 318, Standard Building Code.

Professor Kerekes has been active on ACI technical and administrative committees and is at present a member of the Technical Activities Committee. He is also chairman of the Iowa Building Code Council which recently submitted a proposed "Iowa State Building Code" to the State General Assembly.

Committee 609 changes personnel

Changes in the personnel of ACI Committee 609, Vibration of Concrete, have been announced. A. G. Timms, Senior Materials Engineer, Bureau of Public Roads, Washington, D. C., replaces Frank H. Jackson, also of the Bureau of Public Roads; George W. Washa, University of Wisconsin, Madison, Wis., succeeds Morton O. Withey, Dean of Engineering, University of Wisconsin.

Both appointees have been active on ACI technical and administrative committees. Mr. Timms has had several papers published in the JOURNAL and was awarded the Wason Medal, with N. H. Withey, for "noteworthy research" for the work reported in their 1934 paper "Temperature Effects on Compressive Strength of Concrete." He is also a member of Committee 213, Properties of Lightweight Aggregate Concrete; 604, Winter Concreting Methods; 612, Recommended Practice for Curing Concrete; and 621, Aggregate Selection, Preparation, Handling and Use.

Professor Washa is Secretary of Committee 115, Research, chairman of Committee 213, Properties of Lightweight Aggregate Concrete, author of numerous papers, and member of the Board of Direction. He was a Wason Medalist for "noteworthy research" for work reported in his 1940 paper "Comparison of the Physical and Mechanical Properties of Hand Rodded and Vibrated Concrete Made with Different Cements."

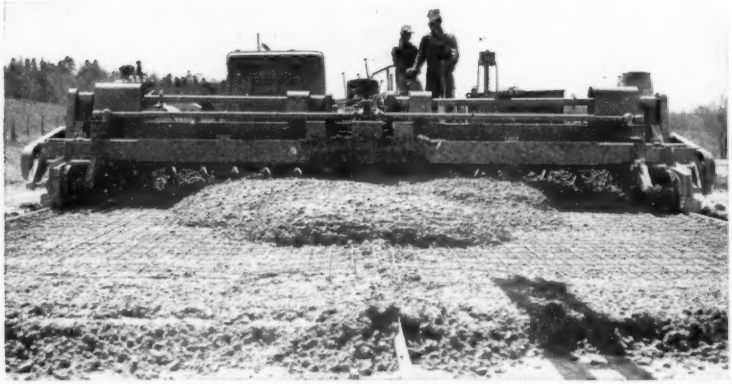
Typical bar bends

A sheet showing typical bar bends, as included in the *Manual of Standard Practice for Detailing Reinforced Concrete Structures* (*ACI Standard 315-48*), has been released by the Institute as an aid to include with specifications, etc. Single copies are available from Institute headquarters at 10 cents each, with special quotations on quantity orders.

Concrete pavement conference

More than 200 engineers, contractors and material suppliers met April 11 at Michigan State College, East Lansing, for a one-day concrete paving conference sponsored by the college and the Portland Cement Assn. C. O. Harris, Head of the Civil Engineering Dept., pointed out that the college did not consider its task complete with the education of the young but that there were continuing responsibilities in research and the spreading of informa-

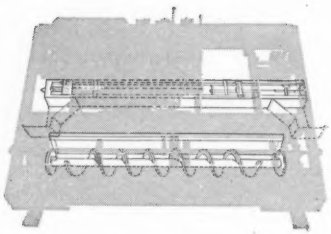
The Spreader and Finisher Team for faster, high-precision paving at Lower Cost!



JAEGER COMBINATION SCREW-SCREED SPREADER

A one-man machine that remixes, spreads, strikes off and then, **by the pass of its 12″ oscillating screed, accurately meters material to the finishing machine.** Insures fast, steady production. Has eliminated second finisher on high production paving. Can complete the concrete base for bituminous surfaced streets. Easily handles the output of two big, dual-drum pavers, eliminates hand labor in the pit, and leaves only precision work for the finisher. Action of its remixing-compacting screw prevents segregation or honeycombing of the concrete, works material against the subgrade and side forms, compacting it into a denser, longer-wearing slab.

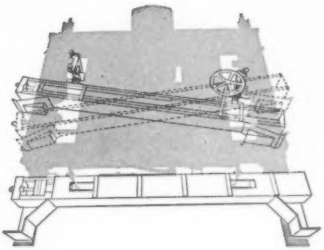
Oscillating metering screed eliminates both excess and deficiency of material ahead of the finisher, insures uniform finish, prevents high spots and the low spots that require costly carryback and interrupt paver production. Finisher can also work farther back from



spreader, permitting concrete to condition sufficiently for better finishing and the finisher to make any extra passes that will improve the work. Working range is to 3½″ below the forms, clearance to 3″ above them. Three forward and reverse speeds, plus high travel speed. Two standard models — SS-10 for 10′ to 15′ widths, and SS-20 for 20′ to 25′ widths. Jaeger Model CS-10 and CS-20 Spreaders are identical except that they are not equipped with screed. Special 15′ to 20′ and 25′ to 32′ widths upon specification.

JAEGER TYPE "X" DIAGONAL SCREED FINISHER

Engineered to match the pace of big dual-drum pavers and Jaeger Combination Screw-Screed Spreaders. Front screed transverse, rear screed pivoted and diagonally adjustable in either direction by the twirl of a handwheel.



Precision finishes because its diagonal screed meets the material at a different angle than the front screed — slices off high spots, fills in low ones. Finishes stiff mixtures fast and without tearing, saves extra passes. **Finishes flush to upper form on pitched slab and elevated curves because the diagonal rear screed moves material uphill and compacts it solidly against the form.** Greatly reduces carry back, saves one or two shovelers. Three working speeds forward and reverse, plus high travel speed. Three screed speeds — independent of traction. Screeds have velvet-touch, spring-cushioned finishing action, hydraulic lift, and single-lever quick-crown-change. Two standard models — for 10′ to 15′ widths and 20′ to 25′ widths, each having 5-foot adjustability in 6″ stages. Special widths — 15′ to 20′ and 25′ to 32′.

There are also Jaeger Type "H" Transverse Screed Finishers in standard 10′ to 15′ and 20′ to 25′ widths — special 15′ to 20′ and 25′ to 32′ widths.

THE JAEGER MACHINE CO., Columbus 16, Ohio

Builders of Paving Machines, Mixers, Truck Mixers, Compressors, Pumps, Hoists and Towers

tion to those already engaged in engineering. J. G. Martin, District Engineer, PCA, said that the conference was born of a desire of road builders to discuss their product and the problems related to its construction.

Max P. Brokaw, Regional Highway Engineer for PCA, discussed "Pavement Construction Practices" and showed slides of improper construction procedures which resulted in pavement failures. He emphasized that a period of increasing replacement of pavements is beginning and that engineers could learn more by finding out why bad jobs were bad rather than trying to discover what made good jobs what they were. Examples of improper joint finishing added weight to his presentation.

In the discussion period following, brief comments were made on the paper, "A Way to Better Pavement Concrete," by Frank H. Jackson which appeared in the March ACI JOURNAL and it was agreed that until some organization engaged in a research program to compare results under present methods with those proposed an exact answer would be unknown.

In considering the relative merits of old-type coarse cements with the newer cements it was mentioned that a test road is being built in Kansas to afford such a comparison. It was remarked, however, that the old-type cement was being ground in modern equipment. Some believe that the absence of the grinding aids formerly used and the difference in equipment might make a true comparison difficult, especially since the old processes may have accidentally induced air entraining properties in the cement.

This led to a discussion of present difficulties with air-entraining cements in entraining less than the specified air contents under some job conditions. Mr. Brokaw pointed out that the specifications for this type of cement have been raised to overcome this and that there was some thought that the specification for the air content of mortar might eventually have to be 19 ± 1 percent to allow for all job conditions. C. H. Cash, Michigan State Highway Dept., said that in Michigan it was optional with the contractor whether the air-entraining agent was ground in the cement or added at the mixer and in those cases where air-entraining cements induced too much air on the job it was blended with normal portland cement to reduce the air content to that specified.

For easier finishing of air-entraining pavement concrete it was suggested that a stiff surface overlaying a rubbery base indicated that the mixture was too wet and that a reduction in mixing water was needed. Retempering finishing machines which added water to the partially set concrete were described with the consensus being that such machines, while producing a smoother finish, would be unacceptable, in this part of the country at least, because they appear to nullify the reduction in mixing water made possible by air entrainment.

In the afternoon session, E. A. Finney, Research Engineer, Michigan State Highway Dept., described in "Pavement Roughness Surveys," the evolution of machinery for measuring surface irregularities which led to the present Bureau of Public Roads' trailer-mounted equipment. Charts illustrated the results from surveys made of pavements laid during recent years. Although the roughness varied considerably between projects, few new pavements were excessively rough.

C. J. Carroll, Michigan Road Builders Assn., C. H. Cash, and W. J. Worth, Wayne County Road Commission, comprised a panel which discussed "Riding Qualities of Concrete Pavements." Mr. Worth said that the inspector often was unfairly blamed for unsatisfactorily finished pavement surfaces. The use of air entrainment, porous subgrades, greater width of lanes with increasing weight on forms, and curing methods were cited as factors contributing to unevenly finished surfaces.

Mr. Cash considered uniform concrete, proper subgrade materials, forms, and mixing time essentials to final pavement smoothness. It was pointed out that the

development of high-capacity mixing and finishing machinery imposed greater loads on forms which necessitated heavier forms and more careful form setting to insure against distortion and displacement. Michigan specifications now establish minimum standards for forms to eliminate them as a factor, so far as possible, in producing poor riding qualities.

The relative merits of keyed joints and deformed tie bars between adjacent lanes in lane-at-a-time construction were considered. Some difficulty was reported with form removal with keyed joints as well as improper vertical placement which resulted in the spalling off of thin upper lugs when the key was too close to the surface. On the other hand, with tied slabs there was some fear that deformation of one slab might cause cracking of the adjacent slab tied to it. It was pointed out, however, that properly placed tie bars of large diameter have been successfully used in airport paving 21 in. thick.

C. J. Olsen, Materials Engineer, Michigan State Highway Dept., briefly discussed "Michigan's Revised Paving Specifications" issued in April. Although this is the first new specification since 1942, supplements since that time had kept the old specification up to date until a new one could be issued. ACI Committee 208, Bond Stress, recommendations for the use of new-style deformed bars are incorporated in the new specification.

Concrete Primer reprinted

The *Concrete Primer* written in 1928 by F. R. McMillan, Honorary Member of ACI, is now available in its sixth printing (a total exceeding 75,000 copies). One of the best and handiest sources of basic information on concrete it can still be rated as a "best buy" although ACI has been forced to increase the price from 25 to 35 cents due to increased printing costs.

Frick re-opens office

Walter A. Frick, after duty with the armed services, has re-established his office as a consulting architectural engineer in Pittsburgh, Pa. The office will specialize in furnishing services as structural engineer for building construction, including the preparation of the design, the working drawings and specifications, and the general supervision and consultation during the erection. Mr. Frick is a member of ACI, A.S.C.E. and A.S.T.M.

French publications

Institute headquarters regularly receives the French-language magazines *La Technique Moderne Construction* and *Revue des Materiaux* for which reviewers are needed. The first publication covers heavy construction methods and equipment while the latter contains more theoretical treatment of the properties of materials. Those interested in regularly reviewing either of these periodicals please write the Secretary for further information.

Wright becomes partner

J. G. Wright has become a partner in the civil engineering firm of Earl and Wright, San Francisco, Calif. Mr. Wright previously held the title of chief engineer in the firm headed by Austin W. Earl.

Bahmeier transferred

H. F. Bahmeier, construction engineer for the Bureau of Reclamation since 1930, has been transferred from Davis Dam to the Delta district of the Central Valley Project. He will direct the Bureau's construction work on the American River development, working closely with the Corps of Engineers, which is building Folsom Dam. Mr. Bahmeier's 20-year career on western reclamation projects has included supervision of construction of the Island Park Dam in Idaho, Vallecito Dam and San Luis Project in Colorado, Davis Dam and power plant on the Colorado River, and early stages of the Anderson Ranch Dam.

Friel named to A.S.C.E. board

Francis S. Friel of Albright and Friel, Inc., consulting engineers, Philadelphia, Pa., has been appointed a member of the

Board of Direction of A.S.C.E., to succeed the late Joel D. Justin. Mr. Friel, an ACI Member since 1920, is a graduate of the Drexel Institute of Technology, and has long specialized in private, municipal and sanitary engineering work, particularly in water supply and purification, sewage collection and treatment, special investigations and reports, public utility rates and appraisals and design and construction. From 1931 to date, he has directed an engineering staff varying from 20 to 300 engineers. During that period he has been associated with more than 1500 projects, of which more than 200 were major engineering undertakings involving a construction cost exceeding $100,000,000.

Elected to membership

Among those elected to membership in the New York Assn. of Consulting Engineers is Jacob Feld. Mr. Feld, an ACI Member, is the author of several JOURNAL contributions.

Stanton Walker Fellowships

The University of Maryland, College Park, Md., offers two Fellowships, sponsored by the National Sand and Gravel Assn. Research Foundation and designated as the Stanton Walker Fellowships. The Fellowships were established in 1940 to honor Mr. Walker, Engineering Director of N.S.G.A. and past president of ACI.

The Fellows start in September, 1950, and continue through June, 1952. The stipend is $750 per year with tuition and other fees paid and text books purchased by N.S.G.A.

Appointees to the Fellowships will register in the Graduate School of the University of Maryland; work will be scheduled so that the time will be divided about equally between study of selected and approved courses and research on appropriate problems related to the sand and gravel industry. Fellowships are open to qualified graduates in engineering from accredited colleges and universities; applications will be received up to June 1, 1950, by the Dean of the Graduate School, University of Maryland.

Mr. Walker, active in ACI affairs since 1921, was president in 1947 and is now on the Board of Direction. He was a member of the Advisory Committee, 1938-46; Standards Committee, 1937- ; Publications Committee, 1945-46; Technical Activities Committee, 1947; Board of Direction, 1940-44; and Vice-President, 1945-46. He is the author or co-author of 12 JOURNAL contributions. He is also a member of five ACI technical committees.

Lone Star erects new plant

The Lone Star Cement Corp. announced recently that it will construct a new cement plant near Roanoke, Va. The plant, to be erected at a cost of $6,000,000, is to have an annual productive capacity of 1,500,000 barrels of cement and should be in operation early in 1951.

The plant will be served by the recently completed Cloverdale and Catawba Railroad, a subsidiary of Lone Star.

With a total annual capacity of 27,500,000 barrels, the Lone Star Cement Corp. is one of the larger cement manufacturers. In addition to ten plants in the United States, its subsidiaries own and operate five plants in Cuba, Argentina, Brazil and Uruguay.

This new plant will bring to 16 the total number of mills in the Lone Star system, with a total annual capacity of 29,000,000 barrels of cement.

A.S.T.M. Committee C-9

During A.S.T.M. Committee Week in Pittsburgh, A.S.T.M. Committee C-9 on Concrete and Concrete Aggregates met and reviewed recent developments. . In the research subcommittees, consideration is being given to new approaches in measuring alkali reactivity; the size and pore spaces in aggregate and their relation to durability; and the development of dynamic testing methods, particularly for field use.

Continued on p. 12

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The Mystic River Bridge, Boston, is paved with Darex AEA concrete to increase resistance to freezing, thawing, and de-icing salt action. Photo courtesy The Boston HERALD.

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Various projects and actions were reported from the specifications and test methods subcommittees. A change was recommended in the *Standard Method of Sampling Fresh Concrete* (C 172) to reconcile the procedures on flow and slump.

What might be considered as a revolutionary move was the decision to include limits in the *Standard Specifications for Concrete Aggregates* (C 33) for flat and elongated pieces.

The conditioning temperature, as well as the mixing water and storage tank temperature, in the *Standard Method of Test for Measuring Mortar-Making Properties of Fine Aggregate* (C 87) was recommended for change to conform to the standard approved temperature of 23 ± 1.7 C. The procedures for analyzing hardened concrete are under study for revision, and a different approach to the procedure is expected.

Kenneth B. Woods, Purdue University, West Lafayette, Ind., is chairman of the committee; Stanton Walker, National Sand and Gravel Assn., Washington, D. C., is secretary.

Applied mechanics program to be held at VPI

Six eminent engineers and scientists will participate in a Professional Engineering Program in Applied Mechanics to be held at Virginia Polytechnic Institute, June 14 to July 22.

Sir Richard V. Southwell will come from England to present his technique of relaxation methods. The preliminary lectures and practical computation periods are designed to provide training in the fundamentals of relaxation technique. Subsequent lectures will apply the methods to problems in elasticity, plasticity, vibrations, elastic stability, fluid mechanics and heat transfer.

The other special feature of the program will be a symposium on modern developments in design in which the following engineers will participate: Hardy Cross, Civil Engineering Dept., Yale University; M. Hetenyi, Professor of Mechanics, Northwestern Technological Institute; L. S. Jacobsen, Head, Mechanical Engineering Dept., Stanford University; R. D. Mindlin, Professor of Civil Engineering, Columbia University; and D. B. Steinman, Consulting Engineer, New York. Each of the men will present five lectures.

The subjects include structural design, experimental stress analysis, transient vibrations of structures, dynamics of package cushioning and aerodynamic loads on bridges and buildings.

Building industry conference

A building industry conference was held at Iowa State College, Ames, Ia., in April to discuss new developments and problems confronting the building industry. The conference was sponsored by the State Building Code Central Committee and the engineering division and engineering extension service of Iowa State College.

The three-day meeting included topics on various phases of building construction, among which were high strength, high bond reinforcing bars; precast concrete construction; concrete pile foundations; thin shell type structures; detailing concrete reinforcement; and reinforced concrete design.

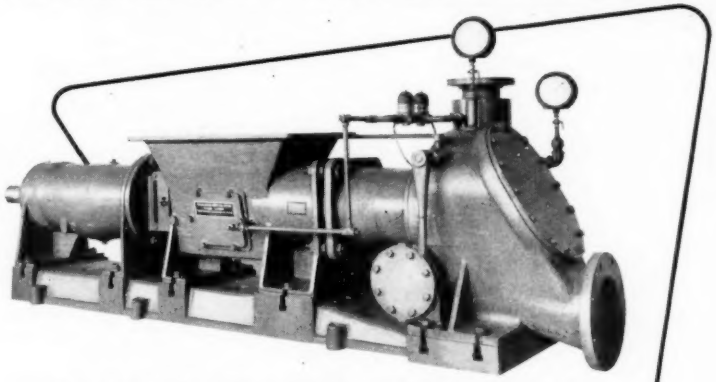
Timber, steel and aluminum construction were also discussed, as well as heating, lighting and other problems.

HHFA research advisory group meets

The first meeting of a Housing Research Advisory Committee was held last month as a part of the program established by the Housing Act of 1949. The 26 men and women, experts in the housing field, will aid the Housing and Home Finance Agency in their program.

"The committee, which covers a broad range of experience in the economic, finance, consumer and technical aspects of housing, will give advice and guidance on the entire housing research program," said HHFA Administrator Raymond M. Foley. "It will also assist in the evaluation of research proposals, and the coordination of housing research activities, both public and private, and aid in the promotion and

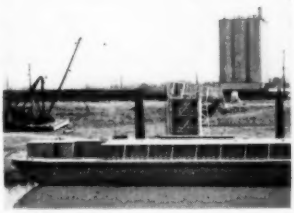
DO YOU KNOW ABOUT PUMPING CEMENT FROM BARGE TO STORAGE? IT'S SIMPLE WITH A FULLER-KINYON PUMP



Here's the modern, economical solution to the problem of conveying bulk Portland cement from barges to storage silos. The Fuller-Kinyon Stationary Pump is installed in the hold; from there, the cement is conveyed direct, in a single continuous operation, to the storage silos.

Bulk cement is delivered to the pump by a drag scraper. Conveyance is fast . . . automatic . . . dust-and waste-proof. Maintenance is extremely low and supervision is virtually unnecessary.

Maybe you've never considered *pumping* pulverized materials in bulk from receiving point to storage. If you haven't . . . you may be passing up a real opportunity to shave your materials handling costs. Fuller Engineers will be glad to consult with you and show you how a Fuller-Kinyon Conveying System can be installed wherever a pipeline can be run . . . how minimum expenditure on your part can result in major savings. Fuller suggestions, based on a thorough survey of your present materials handling system, places you under no obligation whatever . . . may mean a long stride toward higher operating efficiency.



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P-123

application of results of the program, with a view to effectuating cost reductions in housing."

ACI Past President Ben Moreell, president of Jones and Laughlin Steel Corp., is among the members of the committee. Admiral Moreell was formerly head of the Bureau of Yards and Docks, Navy Dept.

Building research congress to be held in London

A Congress on Building Research, the first of its kind, will be held in London, England, Sept. 11-20, 1951, during the Festival of Britain. The congress program will review the progress made in research in relation to architecture, building and associated branches of civil engineering.

The congress is sponsored by the British professional institutions, technical societies and governmental departments, with the support of representative industrial federations in Great Britain.

Papers are being invited from research workers in many countries on a wide range of topics, including: prefabrication, concrete design, soil mechanics and the design of foundations, weathering and durability of building materials, lightweight concrete, quality control and accelerated curing of concrete and many others.

The congress will be organized in three divisions which will hold concurrent meetings; visits to buildings of interest and civil engineering projects will also be arranged.

Highway Research Board meeting

The 30th Annual Meeting of the Highway Research Board of the National Research Council will be held in Washington, D. C., Jan. 8-12, 1951. Headquarters will be the building of the National Academy of Sciences. The change from the Board's usual December meeting dates was necessary, it said, to avoid conflict with other important meetings involving highway engineers and administrators which are scheduled for December, 1950.

Monster "Muscles" passes tests

"Muscles," the world's largest universal testing machine, erected by the U. S. Bureau of Reclamation at its research and testing headquarters in Denver, Colo., passed its initial tests recently by matching strength against a solid steel rod 8 in. in diameter.

The rod gave up the uneven struggle and broke cleanly in two when 4,175,000 lb of pulling power (83,500 psi) were exerted against it—"Muscles" can pull five million pounds.

The mammoth 750,000-lb machine was built and installed by the Baldwin Locomotive Works. The test with the rod was the first of a series of acceptance tests. After the tests are completed, the U. S. Bureau of Standards will check the machine's readings and then "Muscles" will be put to work by the Bureau of Reclamation.

Effect of heavy loads on pavement to be studied

The Highway Research Board announced recently that plans have been completed for a large-scale test of concrete pavement under concentrated truck traffic using single axle loading of 18,000 and 22,400 lb per axle and corresponding tandem axle loadings of 32,000 and 44,800 lb. The relative effects of different axle loads will be tested on parallel lanes of the concrete test road. The trucks will operate at an average frequency of one truck per minute on a 24-hour per day, 7 day per week schedule.

This project has been arranged cooperatively by the states of Connecticut, Delaware, Illinois, Kentucky, Maryland, Michigan, Ohio, Pennsylvania, Virginia, and Wisconsin, and the Bureau of Public Roads, Department of Commerce. Several other states are expected to join the group. The project is to be administered and conducted by the Highway Research Board.

A 1.1-mile section of U. S. Route 301 in southern Maryland has been set aside by the State Roads Commission of Maryland

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for these tests. Traffic ordinarily carried by this road will use an alternate parallel road during the test period. This road consists of two 12-ft lanes of mesh reinforced concrete pavement laid on a good granular subgrade. Each lane is 9-7-9 in. in cross section. It is recognized that the structural character of this road is probably better than that of the average concrete road built for similar traffic conditions. The road was paved in 1941 and has withstood normal traffic and seasonal weather changes for 9 years with little damage.

The 1.1-mile section of road will be divided into two sections. In one section, single rear axle trucks loaded to 18,000-lb axle load will run back and forth over one lane and trucks loaded to 22,400-lb axle load will run back and forth the same number of times in the parallel lane. The other half-mile section will be tested in the same way with trucks equipped with tandem axles loaded to 32,000 lb in one lane and 44,800 lb in the other. In addition to detailed observations of visible effects of the traffic on the pavement, measurements of surface elevations, stresses and deflections under wheel loads, subgrade condition, and concrete quality will be made.

Built-in stress indicators

A new 5-story, 89-room "engineering community center" has been built by the University of Tennessee College of Engineering, Knoxville, Tenn., with stress indicators built into the framework for future observation.

Fourteen SR-4 gages were attached to reinforcing bars in the concrete in the framework during construction and insulated leads extended to contacts inside the building. Working on an electric resistance principle, the gages will reveal some of the changes which take place in the structure as it settles.

Dummy gages, placed near the others but protected so that they will not reflect strain, will make possible a correction for temperature changes.

Honor Roll

February 1 to March 31, 1950

Honor Roll leader tells "how"

Newlin D. Morgan, Professor of Architectural Engineering, University of Illinois, who has led the Honor Roll for two consecutive years and is currently leading in a third year, writes as follows in acknowledging a certificate from the Board of Direction:

At the University of Illinois the American Concrete Institute has not been hard to sell. Our advanced students in architectural engineering have been deeply impressed with the wisdom of keeping abreast of the important field of engineering research and design sponsored by the Institute.

They are interested in the membership personnel and in the papers and discussions presented by scientific men. I am hoping that in the process of time some of them may become contributors in their own right. I was happy to see several recent graduates in attendance at the Edgewater Beach Hotel meetings in February. I look forward to a continuation of this interest at Illinois.

No salesmanship whatever is used. Our graduates know that affiliation with this kind of organization can be nothing but an asset to a young engineer. They want to be informed and they want to get acquainted. This is a logical first step. I am sure the benefit is mutual—both to them and to the Institute.

With his flying start of 34 Member Applicants in February, Professor Morgan easily held top place on the Honor Roll for the period ending March 31. Starting from scratch, Leon Venegas, University of Louisiana, leading member-getter for the month, climbed into second place with credit for 7 Member Applicants.

The life blood of any organization is its active members. The 133 names listed on the Honor Roll represent only 3 percent of ACI's total membership actively encouraging membership growth in the two-month period. Is your name on the list? Are YOU member conscious?

Newlin D. Morgan (Ill.) **34**
Leon Venegas (La.) **7**
R. H. Sherlock (Mich.) **4½**
E. I. Fiesenheiser (Ill.) **3½**
Walter N. Handy (Ill.) **3**
Chester P. Siess (Ill.) **3**
George C. Alden (Calif.) **2½**
Emil A. Gramstorff (Mass.) **2½**
H. J. Gilkey (Ia.) **2½**
Luis A. Pietri-Lavie (Venezuela) **2½**
Roberto Barillas F. (Guatemala) **2**
Henry E. Griset (N. C.) **2**
Wm. T. F. Hooper, Jr. (Ill.) **2**
Stanley Moore (Canada) **2**
S. J. Warberg (Mich.) **2**
Jacob J. Creskoff (D. C.) **1½**
Raymond E. Davis (Calif.) **1½**
Frank H. Jackson (D. C.) **1½**
Melvin W. Jackson (Ga.) **1½**
Cesar Oliver-Rugeles (Venezuela) **1½**
Jose Antonio Vila (Cuba) **1½**
Roderick B. Young (Canada) **1½**
Ralph L. Barbehenn (N. J.) 1
Hugh Barnes (Calif.) 1
J. F. Baxter (N. Y.) 1
Hans H. Bleich (N. Y.) 1
Earl I. Brown (Ga.) 1
Sterling Lowe Bugg (Fla.) 1
W. Fisher Cassie (England) 1
Kenneth W. Clayton (Australia) 1
F. Thomas Collins (Calif.) 1
Clayton M. Crosier (Kans.) 1
Harmer E. Davis (Calif.) 1
Raymond E. Davis, Jr. (Calif.) 1
Charles W. DeGroff (N. Y.) 1
DeWitt Dieterich (Ill.) 1
Belmon U. Duvall (Ohio) 1
George W. Ford (Fla.) 1
William E. Gabelman (Calif.) 1
H. F. Gonnerman (Ill.) 1
Charles E. Hawke (Canada) 1
Terrell R. Harper (Texas) 1
John C. King (Ohio) 1
Leo M. Legatski (Mich.) 1
David V. Lewin (Ohio) 1
Harry E. Lewis (N. Y.) 1
Glenway Maxon (Wis.) 1
James A. McCarthy (Ind.) 1
H. J. McGillivray (Fla.) 1
J. H. Murdough (Texas) 1
J. A. Murlin (Texas) 1
William D. Nowlin (Va.) 1
F. J. Ochoa U. (Guatemala) 1
Wm. Oteovsky (Ill.) 1
Stanley A. Phillip (Ill.) 1
Theodore O. Revhner (Colo.) 1
Evan L. Richard (Australia) 1
Frank E. Richart (Ill.) 1
Horace B. Rickey, Jr. (La.) 1

Continued on p. 24

Who's Who This Month

Committee 208—Bond Stress

Committee 208, Bond Stress, reports committee action on bond stresses in reinforced concrete on p. 677. Herbert J. Gilkey, Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, Ia., is author-chairman. Other committee members are H. P. Bigler, Connors Steel Co.,; Arthur P. Clark, National Bureau of Standards; H. F. Gonnerman, Portland Cement Assn.; Chester L. Post, Consulting Engineer, U. S. Public Buildings Admistration, Federal Works Agency; Raymond C. Reese, Consulting Engineer, Toledo, Ohio; Frank E. Richart, University of Illinois; Chester P. Siess, University of Illinois; David Watstein, National Bureau of Standards; and A. C. Weber, Laclede Steel Co.

Raymond C. Reese

"New-Style Deformed Reinforcing Bars," p. 681, is by Raymond C. Reese, consulting engineer, Toledo, Ohio. An ACI Member since 1936, Mr. Reese is a member of Committee 208, Bond Stress; 318, Standard Building Code; and chairman of Committee 315, Detailing Reinforced Concrete Structures.

Educated in engineering at Massachusetts Institute of Technology and Pierce Hall, Harvard, he spent several years as an industrial engineer with the General Electric Co. Since 1922 he has been in private practice as a consulting engineer; he has been a consultant for the Hausman Steel Co. for 25 years.

Mr. Reese, author of several papers and co-author of one book, is also a lecturer in structural subjects at the University of Toledo. He is the designer of a number of rigid frame structures, including one 540 x 1470 ft, and a specialist in grain elevators and storage bins.

Arsham Amirikian

The author of a number of ACI papers, the latest appearing in the April JOURNAL, Arsham Amirikian, Head Designing Engineer, Bureau of Yards and Docks, Navy Dept., Washington, D. C., submits another paper on precast concrete—"Extent and Acceptability of Cracking in Precast Concrete Framing Members," p. 689. As chairman of ACI Committee 324, Precast Reinforced Concrete Structures, Mr. Amirikian has long been actively interested in precast construction.

After receiving his C. E. degree from Cornell University in 1923, he spent five years with private engineering and fabricating firms. In 1928 he joined the Bureau of Yards and Docks of the Navy Dept. and has been with the Bureau for almost 22 years, beginning as an assistant structural engineer and being appointed to his present position in 1945.

For the past 12 years he has also headed the Bureau's special design section on buildings, floating structures, welding and protective construction. In the latter capacity, he has been associated with most of the important structures built by the Bureau, including the Navy's huge floating drydocks and gate caissons built of reinforced concrete.

In recent years he has devoted most of his time to the technique of precast concrete construction. He developed the thin-shell "hollow-rib" and "cellular" precast concrete framing systems utilized in a number of Navy shore and floating structures. He is presently engaged in adapting these framing systems to protective construction.

George W. Washa and Paul G. Fluck

"Effect of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete," p. 693, by George W. Washa and Paul G. Fluck, presents re-

sults which show the plastic flow characteristics of hand-rodded and vibrated concrete over a 10½-year period.

Professor Washa, Associate Professor of Mechanics, University of Wisconsin, Madison, Wis., is the author of numerous ACI papers. He received the Wason Medal for noteworthy research for the work reported in his 1940 paper "Comparison of the Physical and Mechanical Properties of Hand-Rodded and Vibrated Concrete Made with Different Cements."

He was a member of the Board of Direction, 1948-50; and is now chairman of Committee 213, Properties of Lightweight Aggregate Concrete; secretary of Committee 115, Research; and a member of Committee 609, Vibration of Concrete.

Professor Washa graduated from the University of Wisconsin in 1930 with a B. S. in civil engineering; received the degree of doctor of philosophy from the same institution in 1938. He has been actively engaged in research work for a number of years at the university.

Also at the University of Wisconsin, Professor Fluck is Assistant Professor of Mechanics. He received his B. S. in civil engineering from the university in 1941, and M. S. in 1947, and Ph.D in 1949. He has been on the staff of the Mechanics Dept. since 1941 except for service in the U. S. Navy and short periods with the Wisconsin Highway Commission Materials Testing Laboratory and the Timber Mechanics Division, U. S. Forest Products Laboratory.

Besides being a member of ACI, Professor Fluck is a registered professional engineer in Wisconsin and a member of Tau Beta Pi, Chi Epsilon and Sigma Xi.

Robert F. Blanks

Robert F. Blanks, Chief of the Research and Geology Division, U. S. Bureau of Reclamation, Denver, Colo., reports on "Fly Ash as a Pozzolan," p. 701.

Mr. Blanks, a past president of ACI and presently a member of the Board of Direction, has served on many committees and has contributed frequently to the ACI JOURNAL.

Joining the Bureau in 1925, Mr. Blanks has been with the Bureau's Branch of Design and Construction since 1930. His outstanding work has been the development of laboratory and research techniques to solve diversified engineering problems. The engineering and geological laboratory under his direction has broadened its fields to become one of the largest laboratories of its kind in the world.

Mr. Blanks has been the recipient of many honors and awards from technical and professional societies, including the Thomas Fitch Rowland prize from A.S.C.E. given jointly to Mr. Blanks and Harmon S. Meissner, also of the Bureau. He is also a member of important committees in A.S.T.M. and A.S.C.E.

He has been active also in international professional affairs and has served on subcommittees of the World Power Conference and the International Commission on Large Dams.

Miles N. Clair

"The Inspector," p. 709, by Miles N. Clair, Vice-President, Thompson and Lichtner Co., Inc., Boston, Mass., was briefed at the inspection session of the 46th annual convention in Chicago in February.

Mr. Clair has had numerous papers published in the JOURNAL and, has also been active on both administrative and technical committees of the Institute. He was a member of the Advisory Committee, 1931-39; Standards Committee, 1937-42; and the Board of Direction, 1937-38.

He is a member of Committee 115, Research; 214, Evaluation of Results of Compression Tests of Field Concrete; 318, Standard Building Code; 505, Design and Construction of Reinforced Concrete Chimneys; 604, Winter Concreting Methods; 609, Vibration of Concrete; and 611, Inspection of Concrete. He is also one of ACI's representatives on the

A.S.A. Sectional Committee (A 1) on Specifications and Methods of Test for Hydraulic Cements.

An ACI Member since 1925, Mr. Clair began work on construction jobs at age 11 as a water boy and then as a laborer, mixer man, concrete foreman to construction engineer while he was getting the academic part of his education. He received a B.S. in engineering in 1921 at Drexel Institute and an M.S. in civil engineering from the Massachusetts Institute of Technology in 1923.

From 1923-25 he was an instructor in civil engineering at Drexel Institute; from 1925 to date, he has been testing engineer to vice-president of Thompson and Lichtner Co., Inc.

He is a member of A.S.C.E., A.S.T.M. and the Boston Society of Civil Engineers.

Curzon Dobell

Another paper briefed at the 46th annual convention was "Patents and Codes Relating to Prestressed Concrete," p. 713, by Curzon Dobell, Vice-President, Preload Corp., New York, N. Y. He is the author of many articles on prestressed concrete in American and Canadian magazines; he is also a member of ACI Committee 323, Prestressed Reinforced Concrete.

Mr. Dobell took his engineering training at the Royal Military College of Canada. From 1930-34 he was with the Concrete-Masonry Restoration Co., Ltd., Montreal, Canada, as district manager. In this capacity he made field inspections and engineering reports, especially with regard to the rehabilitation of dams, bridges and harbor facilities.

From 1934-43 he was with Gunite and Concrete Waterproofing, Ltd., a division of the Foundation Co. of Canada, Ltd., first as contract manager and then general manager and director. During this period the company built numerous prestressed concrete tanks and silos including three 100,000 barrel tanks for the Standard Oil Co. at Halifax and a number of alumina silos for the Aluminum Co. of Canada.

Since 1943 Mr. Dobell has been vice-president and director of the Preload companies and the National Gunite Corp. The Preload companies have built over 550 prestressed tanks in the United States alone and also manufacture prestressed concrete pipe. They recently completed a contract for the City of Montreal, Canada, for the largest diameter prestressed concrete pipe built in North America to date; they are also working on the Walnut Lane bridge in Philadelphia, the first prestressed bridge in this country.

Thomas F. Gilbane

Thomas F. Gilbane, President, Gilbane Building Co., Providence, R. I., describes "Precast Concrete Panel Multistory Construction," p. 725. The paper was presented at the ACI New England Regional Meeting in Boston, November, 1949.

A graduate of Brown University, 1933, he has been employed by the Gilbane Building Co. as engineer, general superintendent, secretary of the corporation, and president and treasurer of the company since 1939. The firm was founded by William Gilbane in 1873, with the third generation of the founder's family now managing the company.

The firm has specialized in the construction of institutional and industrial buildings including precast concrete dormitories at the University of Connecticut, a precast concrete housing project at Manchester, N. H., and the General Motors Assembly Plant, Framingham, Mass.

During the war years, Mr. Gilbane served as project manager of the Gilbane Building Co.'s operations at the Naval Operating Base, Newport, R. I. He is a member of the Providence Engineering Society, ACI, past president of the Rhode Island Chapter of the Associated General Contractors of America, and a member of the A. G. C. Endorsements Committee and Quantity Survey Committee.

C. D. Williams and F. Bromilow

C. D. Williams, Head Professor of Civil Engineering, and F. Bromilow, Associate Professor of Civil Engineering, both of the University of Florida, Gainesville, Fla., submit "Tests of Precast Reinforced Concrete Joists," p. 733.

Professor Williams has been in engineering practice and university work for the past 30 years. His various connections include the South Dakota Highway Commission, City of Pasadena, Southern California Edison Co., New York Central Railroad, J. E. Greiner Co., H. K. Ferguson Co. and the J. Gordon Turnbull Co. Since 1945 he has been Head Professor of Civil Engineering at the university. He is the author of several books and numerous articles and is a member of ACI, A.S.C.E., A.R.E.A. and the International Assn. of Bridge and Structural Engineers.

Professor Bromilow received his B.S. in 1937 and his M.S. in 1939 from the University of Pittsburgh. He was on the staff of the civil engineering department as an instructor and assistant professor until 1946. From 1946 to 1948 he was chief engineer of the Plasteel Products Co., Washington, Pa., and joined the University of Florida staff in 1948. Professor Bromilow is a registered professional engineer in Pennsylvania and Florida; he is a member of ACI, A.S.C.E., N.S.P.E., American Society for Engineering Education, Florida Engineering Society, Sigma Tau and Omicron Delta Kappa.

Errata

The following corrections should be made in the paper "A Shortcut for Determining Reinforcement in Reinforced Concrete" by V. Bogvad-Christensen, ACI JOURNAL, Dec. 1949.

p. 286

p = ratio of compression reinforcement to tension reinforcement, should be
Δ = ratio of compression reinforcement to tension reinforcement.
Δ = ratio of compression reinforcement to concrete section, should be
p = ratio of tension reinforcement to concrete section.

$$\tfrac{1}{2}k + p\left(\Delta\frac{f'_s}{f_s} - \frac{f_s}{f_c}\right) = \frac{N}{f_c\,bh} \qquad (2)$$

should be

$$\tfrac{1}{2}k + p\left(\Delta\frac{f'_s}{f_c} - \frac{f_s}{f_c}\right) = \frac{N}{f_c\,bh} \qquad (2)$$

p. 290—Fig. 2—On the section in the lower right corner, dh, is the distance from the tension reinforcement to the top of section and not as shown.

Are you "MEMBER CONSCIOUS"?

New Members

Of the 98 applications for membership approved for the month of March, 58 are Individual, 3 Corporation, 3 Junior and 34 Student.

Canada took top place honors for the month providing 10 new Member applicants. Illinois was close behind with 9 applicants and New York shared third place honors with Louisiana with 7 applicants each.

ACI's membership total March 31, 1950 became 5056.

Alabama

Kirkpatrick, Lester C., (Indiv.) 6808 1st Ave. S., Birmingham, Ala.

Arizona

Earickson, Robert R., (Indiv.) 1729 11th Ave., Yuma, Ariz.

California

Gunn Construction Co., (Corp.) 2903 Beverly Blvd., Los Angeles 4, Calif. (V. S. Lilly)

Kraai, Paul P., (Indiv.) Wishon, Calif.

Martin, J. Edward, (Indiv.) 333 S. Beaudry Ave., Los Angeles 13, Calif.

Pehl, Clifford E., (Indiv.) c/o Bechtel Corp., 220 Bush St., San Francisco 4, Calif.

Prasas, Singheswan, (St.) 1329 Walnut St., Berkeley 9, Calif.

Roselip, Bernard, (Indiv.) 1343 Mill St., San Luis Obispo, Calif.

Connecticut

Marchant, Charles G., (Indiv.) 967 Farmington Ave., W. Hartford 7, Conn.

District of Columbia

Daniels, Walter T., (Indiv.) 772 Kenilworth Terrace N.E., Washington 19, D. C.

Grieb, William E., (Indiv.) Public Roads Administration, Washington 25, D. C.

Werner, George, (Indiv.) Bureau of Public Roads, Dept. of Commerce, Washington 25, D. C.

Florida

Cadenhead, George Lorimer, (Indiv.) 412 S. W. 16th St., Fort Lauderdale, Fla.

Georgia

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Taylor, Mark K., (St.) 1538 Markdan Drive, N. E., Atlanta, Ga.

Illinois

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Engman, Charles A., Jr., (St.) 6930 W. 30th St., Berwyn, Ill.

Ironite Company, (Corp.) 208 S. LaSalle St., Chicago 4, Ill. (Millard A. Rauhoff)

Jirgal, Rudolph Martin, Jr., (St.) 4154 N. Leavitt St., Chicago 18, Ill.

Lenz, Henry M., (St.) 6042 S. Green St., Chicago 21, Ill.

Lewis, Burton A., (St.) 1428 Winnemac Ave., Chicago 40, Ill.

Sims, James R., (St.) P. O. Box 266, University Station, Urbana, Ill.

Wasil, Benjamin A., (Indiv.) 3140 S. Michigan Ave., Chicago 16, Ill.

Weigle, John B., Jr., (Indiv.) 712 W. Burlington Ave., Apt. D, La Grange, Ill.

Indiana

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Kansas

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Kentucky

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Fair, Alberto, (St.) Box 5966, Louisiana State University, Baton Rouge, La.

Hernandez, Eddy N., (St.) Louisiana State University, Box 5966, Baton Rouge 3, La.

Meneses, Carlos Jose, (St.) Box 6860, Louisiana State University, University Station, Baton Rouge, La.

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ten Bosch, Jan, (Indiv.) 19259 Bloom, Detroit 34, Mich.

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Neher, Leslie I., (Indiv.) 2016 E. Lead Ave., Albuquerque, N. M.

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Modell, Jerome, (Indiv.) 20-56 24th St., Astoria, L. I., N. Y.
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Tor, Abba A., (St.) 417 W. 120th St., New York 27, N. Y.
Turitzin, A. M., (Indiv.) 565 W. 139th St., Apt. 43, New York 31, N. Y.
Vought, Laurence M., (Indiv.) 1834 Hone Ave., New York 61, N. Y.

North Carolina

Mackintosh, David M., Jr., (Indiv.) Beam & Mackintosh, Builders Bldg., Charlotte, N. C.

Ohio

Stillson, R. B., (Indiv.) The Procter & Gamble Co., Engr. Div., M. A & R. Bldg., Ivorydale, Cincinnati 17, Ohio
Turzillo, Lee, (Indiv.) Intrusion-Prepakt, Inc., Union Commerce Bldg., Cleveland 14, Ohio

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Rhode Island

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Tennessee

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Texas

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Gimarc, Johnny D., (St.) Box 5451, College Station, Texas
Mayes, Elbert Drew, (Indiv.) 2101 N. St. Paul St., Dallas, Texas
Norwood, George A., (Indiv.) 704 N. Dotsy St., Odessa, Texas
Ries, Edward O., (St.) Rt. 1, Texas Agricultural Experiment Station, Lubbock, Texas
Sanderlin, Ralph C., (Indiv.) 710 S. E. 11th St., Grand Prairie, Texas

Utah

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Washington

Hechtman, Robert A., (Indiv.) 115 More Hall, University of Washington, Seattle 5, Wash.

Wyoming

Toms, Albert, (Indiv.) Heart Mountain, Wyo.
Vernon, John C., (Indiv.) c/o U. S. Bureau of Reclamation, Kortes Dam, Wyo.

Australia

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Canada

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Greget, Charles, (Indiv.) c/o Dewalco, 14 Avenue Hoche, Paris 8°, France

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Ruiz P., Rafael, (St.) Callejon Maravillas No. 11, Guatemala, C. A.
Szarata, Otto W., (St.) Av. Central 135, Guatemala City, Guatemala

Iraq

Nawy, Edward George, (Indiv.) 4/7/1 Aurfaly St., Baghdad, Iraq

Lebanon

Khatib, Munthir Tahir, (Indiv.) P. O. Box 1373, Beirut, Lebanon

Mexico

Lezama, Cortina y Cia S. A., (Corp.) Ramon Guzman 114, Mexico 4 D. F., Mexico (Carlos Lezama)

Panama

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South Africa

Fulton, F. S., (Indiv.) c/σ Concrete Association of S. Africa, Keegan House, cor. Burg & Hout Sts., Cape Town, S. Africa

South Pacific

Doyle, Hugh, (Indiv.) c/o Building & Facilities, Pan American World Airways, Canton Island, S. Pacific

Venezuela

Ayala B., Alfredo, (Jr.) Calle de Los Ranchos No. 4 (Sebucan—Los Dos Caminos) Distrito Sucre, Estado Miranda, Venezuela
Knierim P., Conrado, (St.) Calle Colombia No. 19 N. C., Quinta "Eleonora", Apartado 1933, Caracas, Venezuela
Nunez G., Oscar Alfredo, (St.) Avenida Santiago de Chile, Quinta Anita, Urbanizacion Los Caobos, Caracas, Venezuela
Oswaldo, Rovati B., (St.) Este 6-170, El Conde, Caracas, Venezuela

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Concrete gives better radiation protection

Better protection from radiation is given by a new kind of high-water content concrete developed in Pittsburgh, Pa.

The cost of the new concrete is less than ordinary concrete, and it is easy to make. The iron incorporated in the concrete gives it more shielding ability per cubic foot. Less space is therefore required to give the same protection from radiation.

To find the best possible shield for the 400 million-electron-volt proton sychrocyclotron at Carnegie Institute of Technology, Dr. Edward Creutz and K. Downes tested many types of concrete, including magnetite concrete. Their report appeared in *Journal of Applied Physics* for December.

They reported that material made with magnetite ore, cement and water gave better protection than ordinary concrete. This concrete is also easy to handle although it is not as strong as ordinary concrete.

Although the new concrete is more expensive by the cubic foot, using it results in a saving because it takes less space to give adequate protection.

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Wagon drills

Worthington wagon drills are described in a new bulletin issued by Worthington Pump and Machinery Corp. The 10-page illustrated bulletin describes three models: UMW-40 equipped with a 4-in. cylinder bore; UMW-35 with a 3½-in. cylinder bore; and UMW-30 with a 3-in. cylinder bore. Bulletin H-1200-B36A may be obtained from Worthington Pump and Machinery Corp., Construction Equipment Sales Division, Holyoke, Mass.

N.S.G.A. and N.R.M.C.A. laboratory investigations

A 24-page booklet released by the National Sand and Gravel Assn. and the National Ready-Mixed Concrete Assn. lists the laboratory investigations conducted by the two organizations since 1928. For the investigations since 1945, a brief statement of scope and results is included. Projects summarized include investigations of admixtures, freezing and thawing, alkali reactivity, thermal properties, bituminous mixtures and concrete proportioning.

For further information write Joint Research Committee, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., 1325 E Street, N. W., Washington 4, D. C.

Thermal insulation

A new edition of "Simplified Physics of Thermal Insulation" has been published by Infra Insulation, Inc. The 44-page booklet illustrates the installation of the aluminum accordion insulation manufactured by the firm.

Sections are included on the physics of heat transfer and condensation, laboratory tests of insulation and related subjects. The use of insulation in radiant heating systems is described. A fairly complete chart of thermal insulation values for various materials is featured also.

Copies may be obtained from Infra Insulation, Inc., 10 Murray Street, New York 7, N. Y.

Vibrators

A new concrete vibrator catalog has been released by Wyzenbeek and Staff, Inc. Illustrated, it includes operating data on both vibrators and grinders. Copies may be obtained from Wyzenbeek and Staff, Inc., 842 W. Hubbard St., Chicago 22, Ill.

Honor Roll

Continued from p. 16

Emil Schmid (N. J.) 1
Elwyn E. Seelye (N. Y.) 1
Howard Simpson (Mass.) 1
E. R. Smallhorn (Canada) 1
M. Eugene Sundt (N. M.) 1
Pascal L. Tasio (N. Y.) 1
Wm. H. Thoman (Colo.) 1
H. F. Thomson (Ill.) 1
F. E. Votaw (Mass.) 1
A. E. Wynn (S. Africa) 1
R. A. Zern (Pa.) 1

The following credits are, in each instance "50-50" with another member.

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Willard W. Warzyn
Clement T. Wiskocil
H. D. Worthington

SYNOPSES of recent ACI Papers and Reports

Institute papers of this JOURNAL Volume 21 are currently available at prices indicated. Please order by title and title number.

RECOMMENDED PRACTICE FOR THE APPLICATION OF PORTLAND CEMENT PAINT TO CONCRETE SURFACES (ACI 616-49) 46-1

Price 50 cents (in special covers).

REPORT OF COMMITTEE 616—Sept. 1949, pp. 1-16 (V. 46)

Supersedes 38-30 and 45-18.

This ACI standard establishes recommended practices for appropriate usage, age of concrete, preparation of surface, and the preparation, application and curing of portland cement paint. Three appendixes discuss composition, manufacture and storage, and general characteristics and factors affecting durability.

AN ULTRASONIC METHOD OF STUDYING DETERIORATION AND CRACKING IN CONCRETE STRUCTURES.....46-2

Price 35 cents.

J. R. LESLIE and W. J. CHEESMAN—Sept. 1949, pp. 17-36 (V. 46)

A new method and apparatus for field and laboratory testing of concrete is described. The apparatus called the "Soniscope" was originally designed to detect internal cracks in concrete. It develops pulses of ultrasonic sound in the material and measures the velocity of their transmission through it. This pulse velocity has the unique advantage of being independent of the size or shape of the body under test. Measurements can be made with equal facility in mass concrete, slabs or laboratory specimens.

The existence and extent of internal cracks and the depth of visible surface cracks can be determined by the use of this apparatus.

The velocity has been found, by experiment, to be a reliable measure of the condition of the concrete and is particularly useful in deterioration studies. The dynamic modulus can be calculated from this velocity, and values so obtained are found to agree closely with the results of tests using established methods.

MANUFACTURE OF REINFORCED FOAM CONCRETE ROOF SLABS ...46-3

Price 35 cents.

I. T. KOUDRIASHOFF—Sept. 1949, pp. 37-48 (V. 46)

The Russian type of lightweight concrete described used a rosin-glue emulsion to preserve the air voids before the initial set of the cement. Shrinkage was decreased and strength increased through high pressure steam curing. The autoclave treated foam concrete used in the production of precast industrial roof slabs had a unit weight of 47 lb per cu ft and a compressive strength of over 500 psi. The lightweight slabs, used in a load carrying capacity and as insulation, reduced construction time by 50 percent and costs by as much as 20 percent. Test data on roof slabs and production procedures are also described.

SUGGESTIONS ON CONCRETE FLOOR CONSTRUCTION 46-4

Price 35 cents.

ERNST GRUENWALD—Sept. 1949, pp. 49-56 (V. 46)

The relationship between good concrete floors and the proper selection of cement and aggregates is discussed. Data are cited to emphasize the advantage of coarse-aggregate mixes over cement-sand topping for concrete floors.

USE OF AIR-ENTRAINING CONCRETE IN CANAL LINING 46-5

Price 35 cents.

JOSEPH J. WADDELL—Sept. 1949, pp. 57-64 (V. 46)

This paper presents a discussion based on field observations of the use of air entrainment in canal lining concrete. The Bureau of Reclamation made studies of air-entraining agents for use in the irrigation canal lining for the Friant-Kern Canal on the Bureau's Central Valley Project in California. Results to date indicate that appreciable benefits accrue when an air-entraining agent is used in concrete which is placed and compacted by means of a mechanical slip-form. Care is necessary in adjusting concrete mixes to incorporate entrained air because of the sensitivity to mix changes of concrete for slip-form placement.

THE USE OF PORTLAND-POZZOLAN CEMENT BY THE BUREAU OF RECLAMATION..46-6

Price 35 cents.

ROBERT F. BLANKS—Oct. 1949, pp. 89-108 (V. 46)

The Bureau of Reclamation has made extensive studies of pozzolanic materials, and portland-pozzolan cements are now being used in many of the major structures built by the Bureau. The properties of portland-pozzolan cement that are used advantageously in the production of mass concrete are described.

RESISTANCE OF CONCRETE AND PROTECTIVE COATINGS TO FORCES OF CAVITATION.....46-7

Price 35 cents.

WALTER H. PRICE and GEORGE B. WALLACE—Oct. 1949, pp. 109-120 (V. 46)

A machine used for producing cavitation erosion in the laboratory is described and the results of tests made to investigate the effect of mix proportions, surface treatment, and protective coatings on the resistance of concrete to cavitation are discussed. Through proper use of these, the resistance of concrete surfaces to cavitation erosion may be extended three or four times, but even the best concrete will not resist the forces of cavitation for a prolonged period. Heavy rubber coatings bonded well to the surface of the concrete have proved effective.

VACUUM PROCESSES APPLIED TO PRECAST CONCRETE HOUSES...46-8

Price 35 cents.

K. P. BILLNER and BERT M. THORUD—Oct. 1949, pp. 121-128 (V. 46)

The use of vacuum processes in precast concrete construction simplifies the building of fire-resistant, durable structures designed to permit the maximum use of like units. Vacuum processes have thus far been used in one- and two-story houses and one-story industrial or warehouse structures, but further developments should make possible similar benefits for multi-story construction.

The specific vacuum methods utilized are: (1) extracting excess water from freshly placed concrete prior to set, thereby increasing early strength and enabling early handling of units, (2) holding forms in place by vacuum and (3) handling and placing finished and hardened concrete units by cast-in-place closures formed and quickly hardened by vacuum processes.

The use of these methods results in high strength monolithic construction having exterior and interior finished surfaces, with insulation incorporated into the construction, if desired.

NEW TYPE OF CONSISTENCY METER TESTED AT ALLATOONA DAM....46-9

Price 35 cents.

JAMES M. POLATTY—Oct. 1949, pp. 129-136 (V. 46)

Various mechanical devices have been developed for indicating consistency of concrete since the early days of visual inspection after discharge from the mixer. A new type meter to measure the consistency of concrete while it is being mixed was tested at Allatoona Dam and the operation and results are described.

EFFECT OF MIXING SEQUENCE ON THE PROPERTIES OF CONCRETE....46-10

Price 35 cents.

F. L. FITZPATRICK and W. SERKIN—Oct. 1949, pp. 137-140 (V. 46)

The order in which the constituents of concrete (aggregates, cement and water) are combined in the mixing operation, has a significant effect upon the properties of the concrete as to workability, strength, density, surface finish and absorption. Tests are reported.

BOND OF CONCRETE REINFORCING BARS....46-11

Price 35 cents.

ARTHUR P. CLARK—Nov. 1949, pp. 161-184 (V. 46)

The tests reported were made to compare the resistance to slip in concrete (bond) of deformed bars when tested in beams and companion pull-out specimens, to secure information on the effects of size of bar, the type of deformations on the bars and the strength of concrete on the bond. The bars were cast in a horizontal position in all test specimens. The variables were depth of concrete under the bar, length of embedment of the bar in the concrete, strength of concrete and diameter of bar. Slip of the bar was measured at the loaded and free ends. Three tests were made with 2 in. of concrete under the bar, with 15 in. of concrete under the bar and with 3-in., 12-in., and 16-in. embedments.

Bond strengths for the beams and the pull-out specimens were affected similarly by changes in the geometry of the bars and the bond test specimens. They were greater when the bars were near the bottom than when they were near the top of the specimens. The highest bond strengths were obtained with bars having deformations conforming to suggested requirements for maximum spacing and minimum height and providing ratios of shearing to bearing areas less than 10, usually less than 6.

PERLITE AGGREGATE: ITS PROPERTIES AND USES....46-12

Price 35 cents.

J. JOHN BROUK—Nov. 1949, pp. 185-192 (V. 46)

Synthetic expanded volcanic rock, better known as perlite is a fairly recent addition to the lightweight aggregate field. Its use in concrete is governed by weight, gradation, mixing procedure, cellular structure, strength of cell walls, insulating properties, cost, etc. Air entrainment appears to be necessary to give a workable, nonsegregating mix. Perlite aggregate blended with sand can be used in high strength structural concrete and concrete products.

THIN WALL CONCRETE SHIP CONSTRUCTION....46-13

Price 35 cents.

FRANCIS R. MAC LEAY—Nov. 1949, pp. 193-204 (V. 46)

The development of construction methods for placing thin concrete walls is described. After several attempts using the standard method of pouring between double forms, a new method was developed which permitted the successful construction of ¾-in. and 1½-in. concrete walls. Tests were also conducted to determine the practicability as well as the strength of Gunite as a medium to unite precast units into a monolithic ship. Later these methods were used successfully in the construction of a concrete landing craft for the United States Navy.

SPECIFICATIONS SHOULD BE REALISTIC....46-14

Price 35 cents.

HARRY F. THOMSON—Nov. 1949, pp. 205-220 (V. 46)

Specifications for concrete in moderate-sized and lesser construction frequently contain provisions which are ambiguous, conflicting in application, or do not fully recognize local materials. Most of these questionable features result from (1) inadequate information regarding the characteristics of concrete, or (2) use of ready-written specifications without revision for conditions or changes in standard requirements.

Among the features discussed are: specifying both method and result; "frozen" specifications; habitual use of "1-2-4", multiple provisions for quality; strength without naming consistency; recognition of local materials; use of compression tests; time of placing; bin-test of cement. Numerous quotations from specifications are given, and suggestions offered for bringing the provisions in line with actual conditions.

INSPECTION AND TESTING OF MATERIALS....46-15

Price 35 cents.

NICOLAAS T. F. STADTFELD—Dec. 1949, pp. 237-248 (V. 46)

This paper sets forth the care taken in the inspection and testing of 7 million barrels of portland cement used in 4 million cu yd of concrete of great uniformity and every indication of durability after 12 years of observation. It stresses for the concrete the importance of low water solubility, freedom from "laitance," lack of cement burns of the workers, low alkali content, and it shows above all the necessity for manufacturing control of the cement clinker. It describes how premature set of concrete was prevented. It also deals with the inspection, grading, and testing of aggregates. The Board of Water Supply specifications for aggregates are given as well as other factors used in securing good material.

FLEXURE OF CELLULAR SHELLS...46-16

Price 35 cents.

F. E. WOLOSEWICK—Dec. 1949, pp. 249-256 (V. 46

After a brief discussion of the uses of cellular shells and methods available for the solution of design problems, a sample problem is set up using the theorem of least work. From charts the moments are determined and the effect of added stiffeners is assessed.

CEMENT PERFORMANCE IN CONCRETE EXPOSED TO SULFATE SOILS....46-17

Price 35 cents.

L. A. DAHL—Dec. 1949, pp. 257-272 (V. 46)

The Long-Time Study of Cement Performance in Concrete deals with the performance of portland cements in concrete under various conditions of exposure in the field. Among these conditions is exposure to sulfate soils, that is, to the so-called "alkali soils." This part of the investigation is reported in Chapter 5, in which the complete data are given. The present paper describes briefly the work reported in Chapter 5 and the conclusions which have been drawn. Those readers who wish to study the results in greater detail are referred to the more complete report.

EARLY STRENGTH OF CONCRETE AS AFFECTED BY STEAM CURING TEMPERATURES....46-18

Price 35 cents.

JOSEPH J. SHIDELER and WILBUR H. CHAMBERLIN —Dec. 1949, pp. 273-284 (V. 46)

The testing and results obtained on 990 6 x 12-in. concrete cylinders steam cured for various periods and at temper-

atures ranging from 100 to 200 F are discussed. Strength results are given for ages ranging from 6 hours to 28 days and strengths of companion specimens moist cured at 70 F are given whenever possible. The information presented was obtained for the Bureau of Reclamation for writing specifications covering the steam curing of precast units such as irrigation pipe.

A SHORTCUT FOR DETERMINING REINFORCEMENT IN REINFORCED CONCRETE.....46-19

Price 35 cents.

V. BOGVAD-CHRISTENSEN—Dec. 1949, pp. 285-292 (V. 46)

This paper presents a graph which gives the complete relationship between moments, thrusts, concrete dimensions, reinforcement and resulting stresses for the reinforced concrete members in question.

PRECAST UNITS FOR SHORT-SPAN BRIDGES.....46-20

Price 35 cents.

ROBERT C. HANCKEL—Jan. 1950, pp. 317-328 (V. 46)

Bridge replacement in Lowell, Mass., where minimum traffic interruption was necessary, led to the adoption of precast reinforced concrete units. Available mobile hoisting equipment limited the maximum weight of sections to about 12 tons which permitted small bridges to be precast as complete structures, while for larger bridges, subassemblies were precast and incorporated into the bridge by a cast-in-place deck. The precasting procedures and construction processes are described and illustrated.

INFLUENCE OF SUBGRADES AND BASES ON DESIGN OF RIGID PAVEMENTS.....46-21

Price 35 cents.

KENNETH B. WOODS—Jan. 1950, pp. 329-348 (V. 46)

This paper has been developed from research data, published reports, and experiences gained by observing the performance of both rigid and flexible pavements—particularly as related to subgrade soil textures and the use of base courses. Structural failures of rigid pavements, caused by large concentrations of exceptionally heavy loads, indicate the need for an evaluation of subgrades and bases in determining the most economical design of rigid slabs.

Indications are that the structural capacity of rigid pavements can be improved by the use of location procedures which utilize the best in topographic position and subgrade soil textures. For inferior situations—in regard to position and soils—the use of base courses must be evaluated against the economy of using slabs of increased thickness, more reinforcing steel, or combinations of the two.

It is concluded that it is not entirely feasible, with the present state of knowledge, to standardize rigid pavement design. Rather, the available data indicate that design practices should be developed by regions in which the subgrade soil, availability and type of base course materials, climatic conditions, and traffic volumes and loads are evaluated.

INSPECTION OF MASS AND RELATED CONCRETE CONSTRUCTION.....46-22

Price 35 cents.

LEWIS H. TUTHILL—Jan. 1950, pp. 349-360 (V. 46)

This paper points out that inspection of any kind can be no more effective than that permitted by the specifications and, particularly, by the established job standard of inspection. No distinction is made between the usual activities of inspection and those associated with concrete control since both have the same objective: a serviceable and presentable structure. Details of requirements and procedure to this end are discussed.

EXPERIMENTAL GROUTING INVESTIGATION FOR CHIEF JOSEPH DAM.....46-23

Price 35 cents.

J. M. WELLS—Jan. 1950, pp. 361-376 (V. 46)

This paper describes the development of a method for the control of seepage into the excavation areas at the right abutment, and through the right abutment, at the Chief Joseph Dam project on the upper Columbia River in the State of Washington.

Laboratory studies are presented on various types of grout mixtures, as well as techniques and procedures for drilling in coarse gravel by jetting methods, procedure for grouting pervious gravels and the efficacy of grouting treatment.

USE OF POZZOLANS IN CONCRETE.....46-24

Price 35 cents.

RAYMOND E. DAVIS—Jan. 1950, pp. 377-384 (V. 46)

After stating the characteristics of pozzolanic materials, the effects of replacing with pozzolans part of the portland cement in concrete are considered briefly. Results of tests with fly ash and superfine diatomite are cited to show the possible use of these materials in the East and Midwest where natural pozzolans of the West are not economically available.

CONSTRUCTION OF LONG-SPAN CONCRETE ARCH HANGAR AT LIMESTONE AIR FORCE BASE.....46-25

Price 35 cents.

JOHN E. ALLEN—Feb. 1950, pp. 405-416 (V. 46)

The reinforced concrete arch hangar at Limestone Air Force Base is noteworthy for its size, being one of the largest of this type yet constructed in this country. The superstructure is composed of a thin reinforced arch shell, shaped like an inverted catenary. Construction procedures are described.

VOLUME CHANGES IN SMALL CONCRETE CYLINDERS DURING FREEZING AND THAWING.....46-26

Price 35 cents.

RUDOLPH C. VALORE, Jr.—Feb. 1950, pp. 417-436 (V. 46)

The volume-temperature behavior of small concrete cylinders was observed, using a new mercury-displacement dilatometer, during freezing and thawing cycles in which the range 40 to −20 F was traversed at various rates. Specimens cast from a mix containing 6 bags of portland cement per cu yd, plain and modified by the addition of an air-entraining agent, were tested in air-dry, vacuum-saturated and partially saturated conditions.

Volume-temperature relationships for air-dry specimens yielded uniform thermal expansion data, but those for vacuum-saturated specimens showed departures in the form of transient expansions during freezing, and residual expansions following thawing of the order of 0.8 and 0.4 percent, respectively, a single slow cycle produced decreases in dynamic E exceeding 60 percent.

Much smaller departures were observed for partially saturated specimens (65 to 85 percent of vacuum-saturation) and included, in addition to transient and residual expansions, secondary effects of freezing termed "shrinkage" and relaxation phenomena. The magnitude of the transient and residual expansions appeared to depend upon the rate of cooling and heating, the degree of saturation of the specimen, and, during the slow cycle, upon the degree of supercooling before freezing.

INSPECTION AND CONTROL OF CONCRETE FOR HIGHWAY AND BRIDGE CONSTRUCTION.....46-27

Price 35 cents.

H. W. RUSSELL—Feb. 1950, pp. 437-444 (V. 46)

Inspection procedures and personnel assignments used by the Illinois Division of Highways for large, medium and small jobs are described. The importance of assurance that materials used are adequately tested and approved before shipment, determination of proper proportions by trial mixtures, control of air entrainment, correction of batch weights and proper preparation and testing of concrete specimens are emphasized. The inspection of ready-mixed concrete supplied to small jobs poses a problem still lacking an adequate solution.

SOME APPLICATIONS OF ELECTRIC SR-4 GAGES IN REINFORCED CONCRETE RESEARCH.....46-28

Price 35 cents.

EIVIND HOGNESTAD and IVAN M. VIEST—Feb. 1950, pp. 445-456 (V. 46)

The writers describe some applications of SR-4 gages in reinforced concrete research at the University of Illinois. Special emphasis is given to the problems involved in waterproofing gages attached to steel which is later embedded in concrete.

THE ULTIMATE STRENGTH OF REINFORCED CONCRETE BEAMS...46-29

Price 35 cents.

S. D. LASH and J. W. BRISON—Feb. 1950, pp. 457-472 (V. 46)

The results of tests to failure on 57 small reinforced concrete beams are presented, and it is shown that they can be explained satisfactorily on the basis of a simple plastic theory of design. Formulas are proposed for determining the moments of resistance and the critical percentages of reinforcement.

A WAY TO BETTER PAVEMENT CONCRETE.....46-30

Price 35 cents.

F. H. JACKSON—Mar. 1950, pp. 489-496 (V. 46)

This paper discusses the performance requirements of concrete pavements from the standpoint, primarily, of the quality of the concrete as a material. It is pointed out that the lack of durability of much of our present day concrete may be due to our methods of construction. The writer believes pavement durability could be significantly improved by using a scientifically proportioned mix of dry consistency, well compacted by vibration or tamping, and with close control over aggregate gradation, in place of the oversanded plastic mixtures which we now use. He does not believe that air entrainment is necessarily the final answer to the problem of surface deterioration but that we should seriously consider overhauling our entire construction practice as well as our present methods of controlling the uniformity of aggregate gradations.

Current practice in the construction of concrete pavements and airport runways in Great Britain and past practice in Germany in the construction of the autobahnen are discussed to show that the placing of harsh, dry mixtures with close control of aggregate gradations is entirely feasible from the construction standpoint.

DESIGN AND CONSTRUCTION OF A CIRCULATING WATER INTAKE..46-31

Price 35 cents.

W. S. COLBY—Mar. 1950, pp. 497-508 (V. 46)

The author describes the design and construction of a 250,000 gpm power plant intake. An interesting feature of the pump well cofferdam was the reinforced concrete ring wales, which resulted in savings in operations because the interior of the cofferdam was kept clear for excavating, driving of drilled-in caissons, tremie operations and placing of the pump well concrete.

THE CONTRACTOR'S VIEWPOINT OF INSPECTION.....46-32

Price 35 cents.

DONALD C. ANDREWS and NOMER GRAY—Mar. 1950, pp. 509-512 (V. 46)

The interests of owner and contractor are compared and recommendations are made on specifications, personal qualifications of the inspector, and methods of minimizing friction with the contractor.

CORROSION PROTECTION OF THIN PRECAST CONCRETE SECTIONS.....46-33

Price 35 cents.

D. H. PLETTA, E. F. MASSIE and H. S. ROBINS—Mar. 1950, pp. 513-528 (V. 46)

A new electrical resistance technique for measuring the rate of corrosion of steel reinforcing is described. The method employs a thin ribbon 0.008 x 0.25 in. as the resistance element embedded in 6 x 12 in. thin panels, and a Kelvin bridge sensitive to 0.0001 ohm. The data plotted in dimensionless parameter form enable the *half-life* of concrete to be determined at a comparatively early age. The term half-life is defined as the time required for the cross-sectional area of the reinforcing to decrease by one-half its original value due to corrosion. Six mixes, three water-cement ratios, four exposure conditions and three depths of cover were examined.

DESIGN OF HEXAGONAL BINS...46-34

Price 35 cents.

PAUL ROGERS—Mar. 1950, pp. 529-540 (V. 46)

The analysis of hexagonal bins is presented with a practical example, showing the details of computations for lateral pressures, inward pull, hanging loads, diaphragm action and the design of the walls, beams and columns.

PRECAST CONCRETE IN BRITAIN..46-35

Price 35 cents.

P. G. BOWIE and A. R. COLLINS—Mar. 1950, pp. 541-556 (V. 46)

Precast frames for airplane hangars, garages and farm buildings are discussed. Several systems using precast units in housing construction are described and illustrated. Prestressed precast railroad ties and transmission-line poles are mentioned briefly.

THE ZIG-ZAG COURSE OF CONCRETE PROGRESS.....46-36

Price 35 cents.

HERBERT J. GILKEY—Apr. 1950, pp. 573-580 (V. 46)

Retiring ACI President Gilkey emphasizes that concrete has not become more complex, but rather that as knowledge advances, usually in a zig-zag course, not a straight line, more problems and seemingly contradictory half-truths become evident.

CONCRETING ON THE OTTAWA RIVER PROJECTS OF THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO.....46-37

Price 35 cents.

A. L. MALCOLM and R. B. YOUNG—Apr. 1950, pp. 581-596 (V. 46)

Construction procedures on three Hydro-Electric Power Commission of Ontario dam projects are described includ-

ing the placing of concrete in lifts up to 50 ft, material and concrete handling systems, classes of concrete, sand blending and aggregate production, treatment of joints, concrete proportioning, winter concreting, and concrete control measures. A discussion of the merits of high lifts in mass concrete structures concludes the paper.

PROPOSED DESIGN SPECIFICATIONS FOR TWO-WAY FLOOR SLABS.....46-38

Price 35 cents.

N. M. NEWMARK and C. P. SIESS—Apr. 1950, pp. 597-608 (V. 46)

A new design specification for two-way concrete floor slabs is presented. It is proposed as a replacement for the methods currently contained in Section 709 of the ACI Building Code (ACI 318-47). This new specification is based on analyses of continuous rectangular slabs carrying a uniformly distributed load. Account is taken of the continuity of the slab, of the torsional stiffness of the beams, and of the deflection of the beams.

INSPECTION OF BUILDING CONSTRUCTION.....46-39

Price 35 cents.

LEONARD E. DUNLAP—Apr. 1950, pp. 609-612 (V. 46)

From the viewpoint of an architect, inspection on a job involving architectural and structural concrete encompasses a number of factors. Accurate plans and specifications, testing and developing the concrete mix design, proper erection of forms and placement of concrete, and systematic curing require close attention by a competent superintendent and qualified inspectors.

AUSTRALIAN AGGREGATES AND CEMENTS IN RELATION TO CEMENT-AGGREGATE REACTION 46-40

Price 35 cents.

A. R. ALDERMAN, A. J. GASKIN, R. H. JONES and H. E. VIVIAN—Apr. 1950, pp. 613-616 (V. 46)

A wide variety of Australian aggregates and cements has been examined with a view to estimating the possible incidence of expansive reaction in concrete made from these materials.

Mortar bars were observed over periods up to two years and the results correlated with petrographic examination of the aggregates and chemical composition of the cements. This correlation has shown that in most cases the potential reactivity can be assessed by petrographic examination but that doubtful material requires supplementary mortar tests.

SOME AUSTRALIAN STUDIES ON CEMENT-AGGREGATE REACTION IN MORTAR.....46-41

Price 35 cents.

H. E. VIVIAN—Apr. 1950, pp. 617-624 (V. 46)

This paper summarizes briefly some of the papers on cement-aggregate reaction which have been published by the Commonwealth Scientific and Industrial Research Organization in Australia. These papers deal with four aspects of mortar expansion; the change in mortar tensile strength as expansion occurs, the effects on mortar expansion of alkali mobility, of void space in the mortar and of different storage conditions.

CARBON DIOXIDE AND THE CEMENT-AGGREGATE REACTION 46-42

Price 35 cents.

A. J. GASKIN—Apr. 1950, pp. 625-628 (V. 46)

Spotting and expansion of cement mortars due to alkali-aggregate reaction can be prevented by treatment of the set mortar with carbon dioxide. Active alkali hydroxides, produced by hydrolysis of cement particles, are thereby converted to carbonates, which are inactive in contact with most aggregates. Sufficient "protective" calcium carbonate can be produced throughout the mortar to confer permanent immunity from spotting and expansion, but no appreciable improvement in tensile strength could be obtained. Carbonated mortars were found to have a high degree of dimensional stability.

SPACING OF MOMENT BARS IN PRECAST JOISTS.....46-43

Price 35 cents.

F. N. MENEFEE and H. L. KINNIER—Apr. 1950, pp. 629-636 (V. 46)

Spacing of moment bars in precast reinforced concrete joists in some cases has been as small as $\frac{1}{8}$ in. although it is probable that most manufacturers use $\frac{3}{4}$-in. shear or diagonal tension bars with corresponding spacing of moment bars.

ACI Building Code places a one in. minimum on spacing. Undoubtedly, the regulations were written with monolithic concrete rather than light precast joists in mind. Spacing has no particular significance if all the requirements for bond are met independently of it.

In a properly designed precast joist, theoretical computations show that bond stresses are always less in proportion to the maximum allowed than are any of the other stresses.

The tests herein reported were an attempt to determine whether the artificial reduction of bond area, up to 30 percent, had any effect on the over-all strength of the joist, and to give some indication as to whether $\frac{3}{4}$-in. spacing of moment bars introduced a controlling weakness. While test results on 14 joists, all of the same dimensions, could hardly produce conclusive evidence, the results, along with the service record of precast joists, indicate the $\frac{3}{4}$-in. spacing of moment bars, with $\frac{3}{4}$-in. maximum size aggregate and with all other requirements of the Building Code met, will give a joist which will fail otherwise than in bond, from which it appears that the present one in. minimum in the current Building Code should be thoroughly studied with a view toward modification for precast joists, with well anchored reinforcement—if opinion and tests justify such a change.

PROPOSED SPECIFICATIONS FOR MINIMUM BAR SPACING AND PROTECTIVE COVER IN PRECAST CONCRETE FRAMING MEMBERS...46-44

Price 35 cents.

ARSHAM AMIRIKIAN—Apr. 1950, pp. 637-640 (V.46)

The use of relatively small size aggregates and favorable conditions for quality control make it possible to place reinforcing bars in precast concrete work at closer spacings than those specified or required in conventionally poured-in-place concrete construction. Better quality control and utilization of rich mixtures make it also possible to obtain corrosion protection of reinforcing with relatively thin covers. Recommendations are given for a new basis of specifying bar spacing and cover, in the form of proposed specifications applicable to precast concrete work, together with a discussion of some of the considerations justifying the suggested specifications.

THE PROBLEM OF SLAB DIMENSIONS.....46-45

Price 35 cents.

REPORT OF SUBCOMMITTEE 2, COMMITTEE 325—Apr. 1950, pp. 641-648 (V. 46)

The dimensions of concrete pavement slabs necessary for optimum results depend on and are influenced by a great many factors, economic as well as physical. The factors and their relationship to slab dimensions discussed include strength, elastic and plastic properties, volume change, subgrades, construction conditions and slab thickness, length and width.

EFFECT OF ENTRAINED AIR ON BOND BETWEEN CONCRETE AND REINFORCING STEEL.....46-46

Price 35 cents.

EIVIND HOGNESTAD and C. P. SIESS—Apr. 1950, pp. 649-668 (V. 46)

This paper reports an experimental investigation of the effect of entrained air on the bond properties of the new Laclede and the Hi-Bond bars, each in ½ in. and ¾ in. sizes. Bond properties were studied through 108 pull-out tests with a constant bar embedment of 12 diameters. Bars were cast vertically as well as horizontally; in the latter case, with 6 and 30 bar diameters depth of concrete under the bars. A 5000 psi basic concrete mix was used, with air content as a major variable.

Within the scope of these tests it was found that up to the normal air content of 4 to 5 percent, the bond of all bars in all positions was reduced, in terms of percent, less than the flexural and compressive strength of the concrete. Where more than 5 percent air was entrained, however, the bond of the horizontal bars was reduced rapidly, probably due to rising air.

REPORT OF ACI COMMITTEE 208 ON BOND STRESS.....46-47

Price 35 cents.

REPORT OF COMMITTEE 208—May 1950, pp. 677-680 (V. 46)

A brief review of committee action on bond stresses in reinforced concrete with a tabulation of recommended values for various types of reinforcing bars.

NEW-STYLE DEFORMED REINFORCING BARS.....46-48

Price 35 cents.

RAYMOND C. REESE—May 1950, pp. 681-688 (V. 46)

The development of reinforcing bars from the original plain round or square forms to the present improved deformed patterns is traced through the early work of Withey and Abrams down to the 1949 report of Clark's tests. Noting that A.S.T.M. has adopted a standard for deformed bars, the author emphasizes the need for revision of design specifications to take into account the characteristics of the improved bars.

EXTENT AND ACCEPTABILITY OF CRACKING IN PRECAST CONCRETE FRAMING MEMBERS.....46-49

Price 35 cents.

ARSHAM AMIRIKIAN—May 1950, pp. 689-692 (V. 46)

One of the important questions arising in precast reinforced concrete construction concerns the acceptability of cracked framing members. Owing to the absence of definite guides for inspection, the fabricator often is penalized by unreasonable rejections. As an aid in this matter, a proposal is made for an acceptability clause which gives practical definitions of cracks and specifies limitations for acceptability.

EXTENT OF SUSTAINED LOADING ON COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE.....46-50

Price 35 cents.

GEORGE W. WASHA and PAUL G. FLUCK—May 1950, pp. 693-700 (V. 46)

This paper presents results which show the plastic flow characteristics of hand-rodded and vibrated concrete over a 10½-year period. Three different cements and three different water-cement ratios were included in the test program. The effect of the 10½-year loading period on the compressive strength and the modulus of elasticity is also shown.

FLY ASH AS A POZZOLAN.....46-51

Price 35 cents.

ROBERT F. BLANKS—May 1950, pp. 701-708 (V. 46)

Fly ash is being used in combination with portland cement in the concrete mix at Hungry Horse Dam because of the benefits derived from its use, including lower cost. The effects of the fly ash as a constituent in concrete on the workability, compressive strength, durability, permeability, heat of hydration, volume change, and counteraction of expansive reaction between aggregates and alkalies in cement are discussed briefly.

THE INSPECTOR.....46-52

Price 35 cents.

MILES N. CLAIR—May 1950, pp. 709-712 (V. 46)

The functions of the inspector include cooperation with the contractor in planning for good workmanship and progress, observation of construction operations and the recording of important facts relative to the work, and the enforcement of specifications and good construction practice. The qualifications for the inspector are discussed; he must be experienced, interested, be able to cooperate, be honest and have lots of common sense.

PATENTS AND CODES RELATING TO PRESTRESSED CONCRETE.....46-53

Price 35 cents.

CURZON DOBELL—May 1950, pp. 713-724 (V. 46)

Based on a study of more than 100 domestic and foreign patents and on correspondence with committees of engineers in England, Switzerland, France and Sweden this paper gives a brief account of the more important groups of patents, a synopsis of codes in preparation in the above countries and the problems to be overcome in drafting a prestressed code for domestic use.

PRECAST CONCRETE PANEL MULTISTORY CONSTRUCTION.....46-54

Price 35 cents.

THOMAS F. GILBANE—May 1950, pp. 725-732 (V. 46)

The construction procedure by which precast concrete panels were used in multistory buildings is described. Brief information is given on other structures where similar construction methods were utilized. Because of inherent economies, the author predicts wider use of precast concrete construction.

TESTS OF PRECAST REINFORCED CONCRETE JOISTS.....46-55

Price 35 cents.

C. D. WILLIAMS and F. BROMILOW—May 1950, pp. 733-748 (V. 46)

Tests made as part of the work of ACI Committee 711 to determine the effect of bar spacing on the strength of precast concrete joists are described. The method of quarter-point loading of commercial joists and the results of tests of band specimens are briefly reviewed. It is concluded that the strength of the joists is controlled by the character of the weld used to fasten the stirrups to the main steel and that the test method used indicated no correlation between effective bond area of the steel and the load carrying capacity of the joists.

DISCUSSION

Discussion closed January 1, 1950

Sept. Jl. '49

Recommended Practice for the Application of Portland Cement Paint to Concrete Surfaces (ACI 616-49)—Report of Committee 616

An Ultrasonic Method of Studying Deterioration and Cracking in Concrete Structures—J. R. Leslie and W. J. Cheesman

Manufacture of Reinforced Foam Concrete Roof Slabs—I. T. Koudriashoff

Suggestions on Concrete Floor Construction—Ernst Gruenwald

Use of Air-Entraining Concrete in Canal Lining—Joseph J. Waddell

Discussion closed February 1, 1950

Oct. Jl. '49

The Use of Portland-Pozzolan Cement by the Bureau of Reclamation—Robert F. Blanks

Resistance of Concrete and Protective Coatings to Forces of Cavitation—Walter H. Price and George B. Wallace

Vacuum Processes Applied to Precast Concrete Houses—K. P. Billner and Bert M. Thorud

New Type of Consistency Meter Tested at Allatoona Dam—James M. Polatty

Effect of Mixing Sequence on the Properties of Concrete—F. L. Fitzpatrick and W. Serkin

Discussion closed March 1, 1950

Nov. Jl. '49

Bond of Concrete Reinforcing Bars—Arthur P. Clark

Perlite Aggregate: Its Properties and Uses—J. John Brouk

Thin Wall Concrete Ship Construction—Francis R. Mac Leay

Specifications Should Be Realistic—Harry F. Thomson

Discussion closed April 1, 1950

Dec. Jl. '49

Inspection and Testing of Materials—Nicolaas T. F. Stadtfeld

Flexure of Cellular Shells—F. E. Wolosewick

Cement Performance in Concrete Exposed to Sulfate Soils—L. A. Dahl

Early Strength of Concrete as Affected by Steam Curing Temperatures—Joseph J. Shideler and Wilbur H. Chamberlin

A Shortcut for Determining Reinforcement in Reinforced Concrete—V. Bogvad-Christensen

Discussion closed May 1, 1950

Jan. Jl. '50

Precast Units for Short-Span Bridges—Robert C. Hanckel

Influence of Subgrades and Bases on Design of Rigid Pavements—Kenneth B. Woods

Inspection of Mass and Related Concrete Construction—Lewis H. Tuthill

Experimental Grouting Investigation for Chief Joseph Dam—J. M. Wells

Use of Pozzolans in Concrete—Raymond E. Davis

Discussion closes June 1, 1950

Feb. Jl. '50

Construction of Long-Span Concrete Arch Hangar at Limestone Air Force Base—John E. Allen

Volume Changes in Small Concrete Cylinders During Freezing and Thawing—Rudolph C. Valore, Jr.

Inspection and Control of Concrete for Highway and Bridge Construction—H. W. Russell

Some Applications of Electric SR-4 Gages in Reinforced Concrete Research—Eivind Hognestad and Ivan M. Viest

The Ultimate Strength of Reinforced Concrete Beams—S. D. Lash and J. W. Brison

Discussion closes July 1, 1950

Mar. Jl. '50

A Way to Better Pavement Concrete—F. H. Jackson

Design and Construction of a Circulating Water Intake—W. S. Colby

The Contractor's Viewpoint of Inspection—Donald C. Andrews and Nomer Gray

Corrosion Protection of Thin Precast Concrete Sections—D. H. Pletta, E. F. Massie and H. S. Robins

Design of Hexagonal Bins—Paul Rogers

Precast Concrete in Britain—P. G. Bowie and A. R. Collins

Discussion closes August 1, 1950

Apr. Jl. '50

The Zig-Zag Course of Concrete Progress—Herbert J. Gilkey

Concreting on the Ottawa River Projects of the Hydro-Electric Power Commission of Ontario—A. L. Malcolm and R. B. Young

Proposed Design Specification for Two-Way Floor Slabs—N. M. Newmark and C. P. Siess

Inspection of Building Construction—Leonard E. Dunlap

Australian Aggregates and Cements in Relation to Cement-Aggregate Reaction—A. R. Alderman, A. J. Gaskin, R. H. Jones and H. E. Vivian

Some Australian Studies on Cement-Aggregate Reaction in Mortar—H. E. Vivian

Carbon-Dioxide and the Cement-Aggregate Reaction—A. J. Gaskin

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Proposed Specifications for Minimum Bar Spacing and Protective Cover in Precast Concrete Framing Members—Arsham Amirikian

The Problem of Slab Dimensions—Report of Subcommittee 2, Committee 325

Effect of Entrained Air on Bond Between Concrete and Reinforcing Steel—Eivind Hognestad and C. P. Siess

Discussion closes September 1, 1950

May Jl. '50

Report of ACI Committee 208 on Bond Stress—Report of Committee 208

New-Style Deformed Reinforcing Bars—Raymond C. Reese

Continued on p. 34

ACI publications in large current demand

Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-48)

This standard is of large format, bound to lie flat and presents typical engineering and placing drawings with discussion calling attention to important considerations in designing practice. It was prepared to simplify, speed, and effect standardization in detailing. It is believed to be the only publication of its kind in English. It is meeting wide acclaim among designers, draftsmen and in engineering schools. Price—$3.00, to ACI Members—$1.75.

ACI Standards—1949

218 pages, 6x9 reprinting current ACI Standards: Building Code Requirements for Reinforced Concrete (ACI 318-47); Minimum Standard Requirements for Precast Concrete Floor Units (ACI 711-46), six recommended practices: Use of Metal Supports for Reinforcement (ACI 319-42); Measuring, Mixing and Placing Concrete (ACI 614-42); Design of Concrete Mixes (ACI 613-44); Construction of Concrete Farm Silos (ACI 714-46); Winter Concreting Methods (ACI 604-48); Application of Cement Paint (ACI 616-49); and two specifications: Concrete Pavements and Bases (ACI 617-44) and Cast Stone (ACI 704-44)—ten ACI Standards in one book, $2.50 per copy—to ACI Members, $1.50. The "Detailing Manual" (ACI 315-48) is available separately only at its established price—see above.

Air Entrainment in Concrete (1944)

92 pages of reports of laboratory data and field experience including a 31-page paper by H. F. Gonnerman, "Tests of Concretes Containing Air-Entraining Portland Cements or Air-Entraining Materials Added to Batch at Mixer," and 61 pages of the contributions of 15 participants in a 1944 ACI Convention Symposium, "Concretes Containing Air-Entraining Agents," reprinted (in special covers) from the ACI JOURNAL for June, 1944, $1.25 per copy, 75 cents to ACI Members.

Air Entrainment in Concrete—Book 2 (1947)

A 204-page compilation of information on air entrainment published after Book 1 (1944), including: "Field Use of Cement Containing Vinsol Resin", and "Laboratory Studies of Concrete Containing Air-Entraining Admixtures," by Charles E. Wuerpel, "Entrained Air in Concrete," a foreword and 14 short papers presented at the 1946 Convention; and discussion of the symposium, reprinted from ACI JOURNALS for September 1945, Feb., June and Dec. Part 2, 1946. $2.25 per copy, $1.50 to ACI Members.

ACI Manual of Concrete Inspection (1941)

This 140-page book (pocket size) is the work of ACI Committee 611, Inspection of Concrete. It sets up what good practice requires of concrete inspectors and a background of information on the "why" of such good practice. Price $1.00—to ACI Members 75 cents.

"The Joint Committee Report" (1940)

The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete submitting "Recommended Practice and Standard Specifications for Concrete and Reinforced Concrete," represents the ten-year work of the third Joint Committee, consisting of affiliated committees of the American Concrete Institute, American Institute of Architects, American Railway Engineering Association, American Society of Civil Engineers, American Society for Testing Materials, Portland Cement Association. 140 pages, price $1.50—to ACI Members $1.00.

Reinforced Concrete Design Handbook (1939)

This report of ACI Committee 317 is in increasing demand. From the Committee's Foreword: "One of the important objectives of the committee has been to prepare tables covering as large a range of unit stresses as may be met in general practice. A second and equally important aim has been to reduce the design of members under combined bending and axial load to the same simple form as is used in the solution of common flexural problems."—132 pages, price $2.00—$1.00 to ACI Members.

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Discussion

Continued from p. 32

Extent and Acceptability of Cracking in Precast Concrete Framing Members—Arsham Amirikian

Effect of Sustained Loading on Compressive Strength and Modulus of Elasticity of Concrete—George W. Washa and Paul G. Fluck

Fly Ash as a Pozzolan—Robert F. Blanks

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Patents and Codes Relating to Prestressed Concrete—Curzon Dobell

Precast Concrete Panel Multistory Construction—Thomas F. Gilbane

Tests of Precast Reinforced Concrete Joists—C. D. Williams and F. Bromilow

Over the life-span of a building, maintenance and safety must be considered. The author analyses these and other factors in

Cost of Long-Span Concrete Shell Roofs*

By CHARLES S. WHITNEY†

SYNOPSIS

Reinforced concrete arched shell construction is compared with other types. It is pointed out that when the use of timber, structural steel or concrete is reasonably appropriate, assuming equal skill in design and the same design requirements, the first cost of buildings with long span roofs is ordinarily not greatly affected by the choice of material. This is especially true in the case of buildings in which the structural cost may amount to only about one-third of the total.

However, this is not necessarily true of the annual cost of the project because that is greatly modified by the length of life, and cost of maintenance, repairs, operation and insurance, which are different for the different materials. These latter cost items are difficult to estimate in advance, but, insofar as possible, cost comparisons should be made on an annual basis rather than initial cost.

CONCRETE SHELL FOR LONG SPANS

The concrete shell construction discussed is a newly developed type which has been used recently in several structures.[1,2] It is especially suitable for such projects as hangars, sports arenas and other auditoriums and can readily provide clear spans of 400 to 500 ft if desired. It consists of a concrete shell from 3 to 5 in. thick, stiffened by concrete arches monolithic with the shell and spaced 20 to 30 ft apart.

The arch ribs are braced laterally by the shell and are made as shallow as practical to reduce the stresses caused by volume changes due to shrinkage, rib shortening, plastic flow and temperature variations. The ribs must also be designed to resist eccentricity of thrust due to deflections caused by critical loading conditions. This eliminates danger of buckling. Since some effects, such as volume change moments, are increased by increasing the rib depth while others such as live load

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†Member American Concrete Institute, Partner, Ammann and Whitney, Consulting Engineers, New York and Milwaukee.

and deflection stresses are decreased, the depth of the rib is proportioned so as to give maximum economy under critical loading combinations.

Hingeless arches monolithic with the abutments are used because hinges are expensive and greatly reduce the stability of the arches. Relief from excessive volume change moments or the effect of abutment movements can best be secured by rib flexibility obtained by slenderness rather than by hinging.

The method of design of the arches will be covered in a forthcoming report of ACI Committee 312, Plain and Reinforced Concrete Arches, supplementing previous reports,[3,4,5] and will not be further discussed here.

The design of cylindrical concrete shells will be treated comprehensively in a report of A.S.C.E. Subcommittee on Thin Shell Design.

The shell used between the arch ribs has a span which is very short compared to its radius and it acts largely like a portion of a cylinder with elastic stiffening ribs.[6] The load is carried for the most part by direct compression and bending from rib to rib, and membrane shearing stresses are low except near the ends of the arch span where shear transmits load to the ribs and the edge beams.

An important feature of this type of construction is the placing of the shell near the neutral axis of the ribs so that the ribs project about half above and half below the shell. The principal effects of this arrangement structurally are the elimination of edge stresses in the shell due to rib flexure and the reduction of the stiffness of the combined rib and shell with a corresponding reduction in volume change moments.

If the shell is placed at the top or bottom of the ribs, it must participate like the flange of a T-beam in the flexure of the rib, and due to the curvature of the shell the tensile or compressive stresses induced along the edges of the shell will produce cross bending as shown in Fig. 1. This bending may be of the same magnitude as the dead and live load bending in the shell and would therefore increase its cost and weight.

Also, placing the shell at the top or bottom of the ribs approximately doubles the effective moment of inertia of the combined shell and rib, with a corresponding effect on the volume change moments which are proportional to the moment of inertia. Since the moments due to volume change and abutment rotation may be almost as great as the live load moments on a structure of this type, it is evident that the

Fig. 1—Cross bending in slab due to rib flexure

location of the shell with respect to the rib can greatly affect the economy of the construction. The following data from the design of a 220-ft span indicate the importance of this feature (Fig. 2).

With the slab at mid-height, the moment of inertia of the crown section is 58,000 in.^{4}. Under a live load of 30 psf the maximum live load bending moment is 1,710,000 in.-lb, and the moment due to volume changes is 1,026,000 in.-lb. The latter was calculated in this case for rib shortening and a temperature drop of 40 degrees F including the effect of shrinkage. This is not sufficient allowance for temperature change in a northern climate. With the slab at the top of the rib, and 70 percent of the slab effective, the moment of inertia becomes 116,300 in.^{4}. The live load moment is the same, but the volume change moment is increased to 2,042,000 in.-lb. The total bending moment is increased from 2,736,000 in.-lb to 3,752,000 in.-lb and the rib is no longer able to carry the load. It must then be increased in size with further increase in volume change effects. This results in a greater dead load and greater cost of ribs, abutments and foundations. The calculations indicate that for equivalent strength the crown section of the ribs must be increased from 18 x 32 in. to 27 x 32 in. with corresponding increase in reinforcing steel when the slab is moved from the mid-position to the top or bottom of the rib. The maximum horizontal thrust against the abutment is increased from 447,120 lb to 508,300 lb. The more flexible design has the advantage of greater economy and may be feasible in cases where the stiffer design is not practical regardless of cost. It has special advantage in the north where temperature changes are greater.

To study the magnitude of this effect, complete designs and cost estimates were made for three twin-span hangars with the shell near the center of the ribs and with the shell at the bottom of the ribs. The spans were 300 ft long and 185 ft deep. Both designs were based on identical conditions of live and wind loads, temperature changes, foundation conditions, allowable unit stresses and factors of safety.

	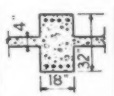	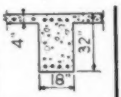	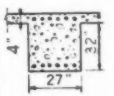
Moment of inertia of crown section	58,000 in.^{4}	116,300 in.^{4}	154,300 in.^{4}
Moment due to live load	1,710,000 in.lb.	1,710,000 in.lb.	1,710,000 in.lb.
Moment due to volume change	1,026,000 in.lb.	2,042,000 in.lb.	2,670,000 in.lb.
Total moment	2,736,000 in.lb.	3,752,000 in.lb.	4,380,000 in.lb.
Maximum horizontal thrust	447,120 lb.	450,170 lb.	508,300 lb.

Fig. 2—Effect of position of slab on bending moment in rib

The summary of costs given in Table 1 includes the roof and supporting structure only, no excavation, floor, walls, doors, masonry work or utilities being included since they are not affected.

The falsework and formwork items include the cost of erecting, moving and dismantling. Although the projection of the rib below the shell increases the cost of the falsework and lower formwork because it is cut by the rib forms and must be lowered farther to clear the rib projection, this is offset by the reduced cost of repeated setting, aligning, holding, and moving the much smaller upper rib formwork.

Observations on several existing thin-shell structures indicate the importance of completely waterproofing and insulating the exterior surface of the ribs and shell. When this is done, greater projection of the rib above the slab increases the cost of that item.

In this case the estimated increase in cost of the structural portions included was about 30 percent due to placing the shell at the bottom instead of at the center of the rib, and this differential would be increased if foundation conditions were unfavorable.

COMPARATIVE ESTIMATES OF COST

Plans have been prepared for an athletic practice building for the University of Wisconsin with a clear floor area 200 ft wide and 400 ft long. Fig. 3 is a preliminary sketch showing the general arrangement of the building.

To determine the most suitable construction for the roof, detailed estimates were made of the cost of six different types in timber, steel and reinforced concrete. These estimates are based on quantities computed from accurate preliminary designs, at unit prices verified by recent costs.

The comparison is based on a clear floor area of 190 ft by 400 ft because that is the narrowest building which will provide suitable practice

TABLE 1—ESTIMATED COST OF THREE TWIN-SPAN HANGAR STRUCTURES*

Construction item	Type A, shell near centerline rib	Type B, shell at bottom of rib
Roof shell (formwork not included)	$ 288,000	$ 330,000
Arch ribs (formwork not included)	348,000	624,000
Falsework, traveler and lower formwork	213,000	190,000
Upper formwork for ribs	31,000	74,000
Buttresses	162,000	228,000
Footings and foundations	453,000	549,000
Insulation and roofing	261,000	324,000
Total	$1,756,000	$2,319,000

*300-ft spans, 185 ft deep.

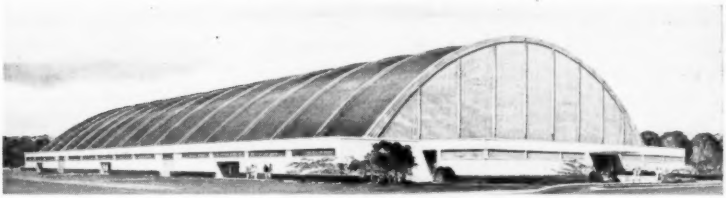

Fig. 3—Athletic practice building, University of Wisconsin

facilities and it is the widest span which can be built using the type of prefabricated steel building with which comparison is desired.

In the case of arched types, a vertical clearance of 60 ft at the centerline of span and 15 ft at the side walls was provided. Type B, in which the roof is carried by timber trusses with a horizontal bottom chord as in the case of the new University of Minnesota building, provides a vertical clearance of 40 ft over the entire area except for the necessary knee braces near the side walls.

The cost estimates include only those items subject to variation with the different types of construction, namely: (1) main arches or trusses, (2) purlins and sheathing, (3) sag rods and bracing, (4) abutments and foundations, (5) additional end and side wall areas required in the different layouts and (6) roofing and insulation.

The types of framing considered are given in Table 2. It should be noted that the arched types of roof provide additional space between the buttresses which will be available for storage of bleachers and other equipment or for other purposes. The most economical spacing of buttresses varies with the different types and this variation is so great as to affect seriously the usefulness of these areas.

Type A consists of laminated glued timber arches 18 ft on centers supporting wood joists 24 in. center to center with 1-in. wood sheathing, insulation and roofing. Abutments and footings are reinforced concrete.

Type B has bowstring timber trusses 16 ft on centers, wood joists, 1-in. wood sheathing, insulation and roofing. The trusses are similar to those used in the new building for the University of Minnesota and

TABLE 2—TYPES OF FRAMING CONSIDERED IN COST COMPARISONS

Type	Main framing	Spacing on centers
A	Laminated timber arches	18 ft 0 in.
B	Bowstring timber trusses	16 ft 0 in.
C	Arched timber trusses	28 ft 6 in.
D	Arched steel trusses	13 ft 6 in.
E	Rolled steel beam arches	28 ft 6 in.
F	Reinforced concrete arches	28 ft 6 in.

the estimate is based on the cost of that building as given by the Timber Engineering Co. The trusses are composed of a laminated timber curved top chord with a horizontal timber bottom chord.

Type C is the most economical timber construction and has 8 ft deep arched trusses on 28 ft 6 in. centers braced with timber sub-trusses carrying wood rafters, 1-in. wood sheathing, insulation and roofing. Buttresses and footings are reinforced concrete. This type has been used by the U. S. Navy for large blimp hangars.

Type D is framed with prefabricated steel arched trusses. To utilize these trusses and provide the necessary vertical clearance of 15 ft at the side wall of a 190-ft span, it is necessary to support them on reinforced concrete buttresses.

These trusses were designed for a live load of 20 psf. To increase their capacity to 30 psf as required by the Wisconsin State Building Code, the fabricator advised that they proposed to place their arches at 13 ft 6 in. centers. The estimate of cost is based on that spacing.

A detailed analysis was made of the stresses in these trusses when spaced 13 ft 6 in. on centers and it was found that they do not have sufficient strength if subjected to a live load of 30 psf distributed so as to produce maximum stresses in the various sections. A building of this importance to be used for public assembly should be designed for such loading conditions and not based on arbitrary location of live load as used for these trusses. An allowance was therefore made for the cost of reinforcing the trusses where necessary. They might be used without such reinforcement by decreasing the spacing still further but it would be even more uneconomical and undesirable to have buttresses closer together. The 13 ft 6 in. spacing makes the space between them unavailable for storage of bleachers and anything but minor equipment. The estimate of cost is based on prices quoted by the fabricator.

Type E is the cheapest type of steel roof for this building, and consists of arches built of rolled beams at 28 ft 6 in. centers, steel beam purlins, sheet steel decking, insulation and roofing, concrete abutments and footings. The steelwork is not encased in fireproofing.

Type F is a reinforced concrete arched shell with concrete ribs spaced 28 ft 6 in. The ribs project 11½ in. below the shell and about the same distance above. Insulation and roofing cover the shell and ribs so that no concrete is exposed to the weather. This type was used for the twin-span hangar built for American Airlines at Chicago with clear spans of 257 ft. It is the cheapest fire-resistant construction which can be used for this building. This type is economical for a building 400 ft long because the formwork for the roof can be used seven times. The construction time for the 400-ft barrel would be about 12 weeks after preparation is made for the first roof pour.

The costs given in Table 3 cover the items previously listed and do not include contractor's profit and overhead charges.

Conclusions

The following conclusions were reached:

Timber construction is little cheaper in first cost than steel for this building, maintenance cost and depreciation would be much higher, and it has nothing to recommend it.

The most economical and satisfactory steel construction is Type E with arches built of rolled beams; it should therefore be considered in comparison with Type F. Type D is undesirable for the reasons stated. If protected by fireproofing, Type E will cost more than Type F.

Type F in reinforced concrete has numerous advantages over Type E and it should actually result in less cost over a period of years although the first cost appears slightly higher. The indicated differential in first cost might be about 2 percent of the total cost of the building but the maintenance cost will be lower than that of steel which will need repainting and will have a greater tendency to cause condensation. The use of concrete also removes the fire hazard and improves the appearance of the interior of the building. It will present a clean, smooth ceiling, free of trussing and bracing, and will make the lighting more effective.

The reinforced concrete arched shell construction, Type F, is most suitable for this building. While the estimates are based on a clear span of 190 ft, the cost of Type F construction per sq ft of floor is not greatly affected by increase in span length and the advantages over other types in its use increase with longer spans.

AMERICAN AIRLINES HANGAR

The American Airlines Hangar,[7,8] completed in 1947, is shown in Fig. 4 and 5. This has two spans of 257 ft each and is 185 ft deep. The concrete roof shell is 3½ in. thick in general, stiffened by arch ribs spaced 28 ft 11½ in. which are 38 in. deep at mid-span. Foundations are spread on Chicago clay.

The construction cost of the hangar complete, without utilities, was $9.34 per sq ft or $0.212 per cu ft. The utilities cost $3.54 per sq ft

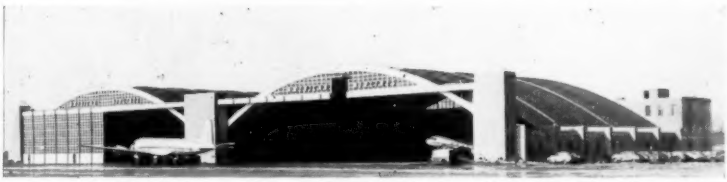

Fig. 4—American Airlines hangar, Chicago, Ill.

TABLE 3—COMPARATIVE STRUCTURAL COST ESTIMATE—UNIVERSITY OF WISCONSIN PRACTICE FIELD HOUSE

ITEM \ FRAMING TYPE		WOOD			STEEL		CONCRETE
		LAMINATED RIB	BOWSTRING	ARCHED TRUSS	ARCHED TRUSS "PREFABRICATED"	ROLLED BEAM	
SECTIONS							
MAINTENANCE		HIGH	HIGH	HIGH	MEDIUM	MEDIUM	VERY LOW
FIRE RATING		NON-FIREPROOF	NON-FIREPROOF	NON-FIREPROOF	NON-FIREPROOF	NON-FIREPROOF	FIREPROOF
SUPERSTRUCTURE	ROOFING	88,000 S.F. @ .25 = $22,000	82,000 S.F. @ .25 = 20,500	97,600 S.F. @ .25 = 24,400	91,840 S.F. @ .25 = 24,400	88,000 S.F. @ .25 = 22,000	91,600 S.F. @ .25 = 22,800
	INSULATION	88,000 S.F. @ .15 = $13,200	82,000 S.F. @ .15 = 12,300	97,600 S.F. @ .15 = 14,640	91,840 S.F. @ .15 = 13,776	88,000 S.F. @ .15 = 13,200	91,600 S.F. @ .15 = 13,700
	SHEATH, PURLINS, RAFTERS, BRIDGING	76.5 M @ 180 = $13,150 77 M @ 180 = $17,500	150 M @ 180 = 27,000 95 M @ 180 = 17,100	67.6 M @ 220 = 14,850 101.5 M @ 180 = 18,300	91,800 F.B.M. @ .30 = 27,600 64 T @ 350 = 22,400	88,000 F.B.M. @ .35 = 30,800 31 T @ 300 = 9,300	
	SUBFRAMING, BRACING		35 M @ 350 = 12,300	136 M @ 280 = 38,200	64 T @ 350 = 22,400	188 T @ 300 = 55,500	
	CONCRETE						1514 C.Y. @ 35 = $52,990
	REINFORCING STEEL						123 T. @ 200 = 24,600
	FORMWORK & TRAVELER						ORIG. + 7 MOVES = 78,200
	RIB FRAMES	169 M @ 625 = 105,000	76,000 F.B.M. @ 1.37 = 104,200	111 M @ 450 = 50,000	250 T @ 300 = 75,000	175 T @ 300 = 52,500	
	ADDED WALL		29,750 S.F. @ 2 = 41,500	9440 S.F. @ 2 = 18,000			
	TOTAL	$176,850	$234,900	$178,390	$184,136	$183,300	$192,290
SUBSTRUCTURE	CONC. ABUTMENTS	320 C.Y. @ 20 = 6400		510 C.Y. @ 20 = 10,200	415 C.Y. @ 20 = 8300	460 C.Y. @ 20 = 9200	592 C.Y. @ 20 = 9200
	CONC. FOOTINGS	405 C.Y. @ 15 = 6075	150 C.Y. @ 17 = 2550	320 C.Y. @ 15 = 4,800	526 C.Y. @ 15 = 7890	320 C.Y. @ 15 = 4800	425 C.Y. @ 15 = 6375
	ABUTMENT FORMWORK	17,000 S.F. @ 1.00 = 17,000		16,980 S.F. @ 1.00 = 16,980	21,900 S.F. @ 1.00 = 21,900	14,820 S.F. @ 1.00 = 14,820	15,300 S.F. @ 1.00 = 15,300
	REINFORCING STEEL	99 T @ 200 = 19,800	3.5 T. @ 200 = 800	44 T @ 200 = 8,800	128 T @ 200 = 25,600	44 T @ 200 = 8800	49 T @ 200 = 9,800
	EXCAVATION	2000 C.Y. @ 2.30 = 4600	345 C.Y. @ 2.30 = 800	1134 C.Y. @ 2.30 = 2,613	2170 C.Y. @ 2.30 = 4990	1134 C.Y. @ 2.30 = 2613	1290 C.Y. @ 2.30 = 2967
	TOTAL	$53,875	$4150	$43,393	$68,680	$40,233	$46,282
TOTAL COST		$230,725	$239,050	$221,783	$252,816	$223,533	$238,572
UNIT COST (TOTAL PER SQ. FT.)		$3.04	$3.20	$2.92	$3.32	$2.94	$3.14

NOTE: ALL FIGURES ARE EXCLUSIVE OF CONTRACTORS PROFIT, INSURANCE, AND CONTINGENCIES

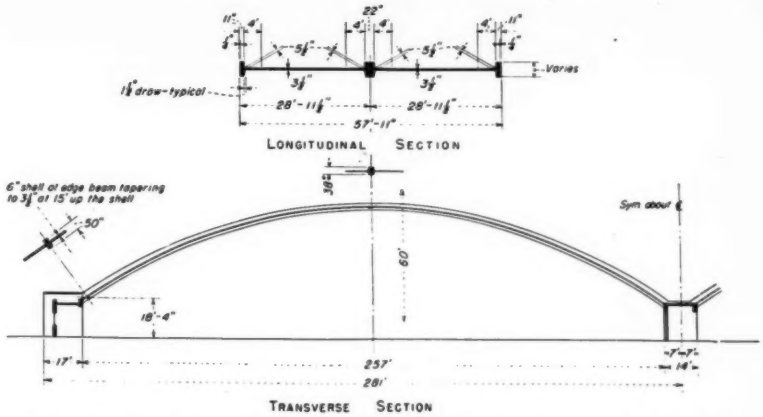

Fig. 5—Sections through one hangar shown in Fig. 4. The shell is generally 3½ in. thick and the ribs are only 38 in. deep at the crown.

including heating, lighting, plumbing, deluge sprinkler system, etc. The size of the structure permitted the use of the roof forms and falsework six times and reduced the cost of building, moving and demolishing the falsework and forms for the roof to about $1.00 per sq ft of floor area.

Before the contract for the concrete structure was awarded, alternate designs were made for several types of steel framing for the roof and comparative estimates indicated that steel construction with a light steel roof deck would cost practically as much as the concrete construction, and that fireproofed steel construction would be considerably more expensive.

ALABAMA LIVESTOCK COLISEUM

The livestock coliseum now under construction for the State of Alabama at Montgomery is unique in design with a circular plan 340 ft in diameter spanned by a cylindrical concrete shell roof 286 ft between supports (Fig. 6 and 7).

Bids were taken in February 1949 on the reinforced concrete shell design prepared by Ammann and Whitney, and on an alternate steel design prepared by the architects. The steel framing, not fireproofed, consisted of plate girder arches with I-beam purlins and was designed for dead load, temperature change, and a 20 psf live load with no consideration of wind loads nor deflection stresses. The concrete design was figured for the heavier concrete dead load, 20 lb live load, heavy vertical and horizontal wind loads, shrinkage, rib shortening, temperature change, abutment movements, and deflection effects.

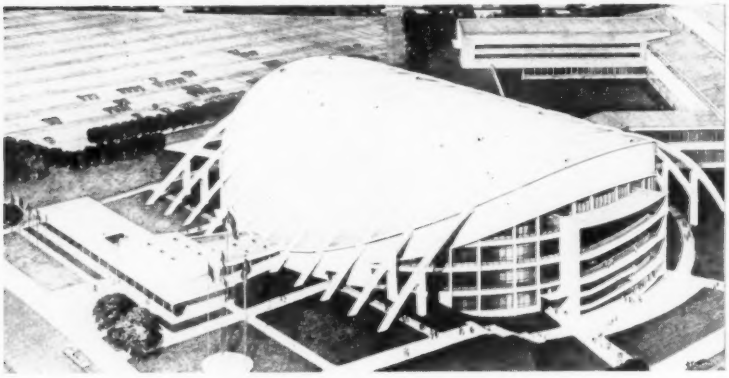

Fig. 6—Alabama livestock coliseum

The low bid on the concrete design for arch roof structure and its supporting frames and footings was $577,500, and for the steel design with a "porex" roof deck $579,000. The time of completion for the concrete structure, including the enclosed building, was 500 days as compared with 730 days for the steel structure. The cost of the concrete roof is $6.34 per sq ft of the 340 ft circular area, including roofing and insulation. If the steel design had been equal in strength to that of the concrete design, the bid price for steel construction would have been substantially higher.

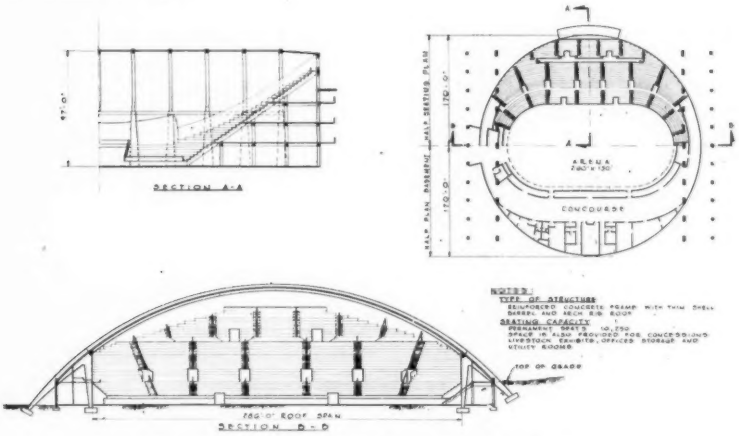

Fig. 7—Sections through Alabama livestock coliseum.

ONONDAGA COUNTY WAR MEMORIAL

The general-purpose auditorium building under construction in Syracuse, N. Y., is also a unique concrete shell construction (Fig. 8). To keep the roof of the auditorium as low as possible and at the same time provide the desired clearances over the balconies, the arched shell is supported on cantilevers. This arrangement is economical because it lowers the thrust line of the arches and reduces the bending at the base of the piers. The clear span of the roof is 204 ft. Foundations are on piles in deep soft clay. Horizontal thrust on the footings is eliminated by tying across through the first floor construction. The structure is of reinforced concrete throughout. Preliminary designs and estimates indicated that the concrete roof construction was cheaper than an unfireproofed roof with steel rigid frames which would give about the same clearance below.

The cost of the architectural and structural portions of the project is \$0.539 per cu ft and the utilities, including heating, ventilating, lighting, plumbing, and skating rink refrigeration and piping, is \$0.153 per cu ft. The structural cost, including the pile foundations, is \$0.245 per cu ft.

EFFECT OF SPAN LENGTH ON COST

Fig. 9 indicates the results of a study to determine the effect of span length on the cost per sq ft of concrete arched shell roofs. The cost includes the entire structure of a typical interior bay without end walls. It is based on 15 ft vertical clearance at the side wall and a total of six uses of the falsework. It indicates that an increase of 100 percent in span from 150 to 300 ft increases the cost per sq ft only 15 percent.

Within ordinary limits the cost per sq ft may be affected more by the length of the building than by the span length because of the influence of the cost of end walls and the number of times the falsework can be used. The cost is also influenced by the extent to which the construction work can be organized to keep various kinds of labor

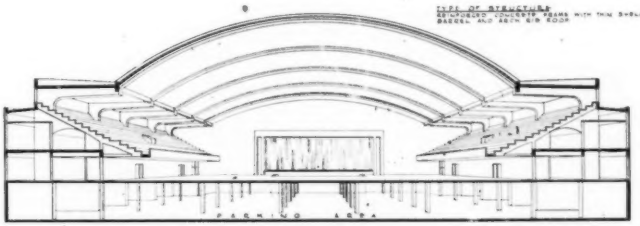

Fig. 8—Interior perspective of Onondaga County War Memorial

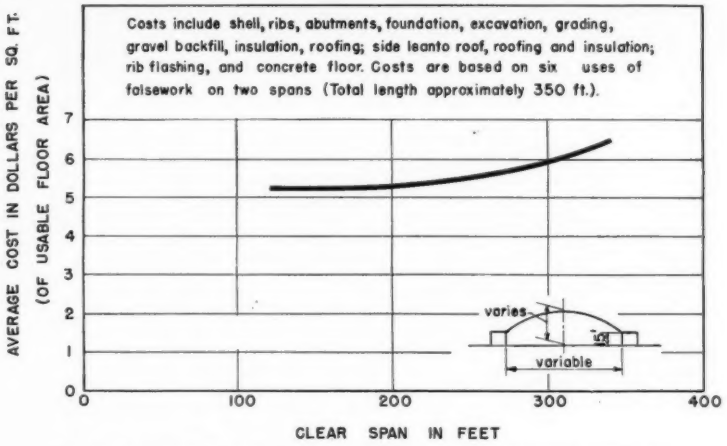

Fig. 9—Variation of cost with span in reinforced concrete barrel construction

continuously occupied. A long building permits the rotation of crews on the building of footings, abutments, roof spans and lean-tos.

For these reasons, while it may be said that in general the construction cost of reinforced concrete arched shell structures should not be widely different from other types, an accurate comparison requires a careful study for each particular case.

Because the structural cost is only a portion of the cost of a project and because the annual cost is influenced by other things in addition to first cost, the selection of type may depend largely on depreciation, maintenance and operation costs and general adaptability of the type to the purpose of the project.

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Discussion of a paper by Charles S. Whitney:

Cost of Long-Span Concrete Shell Roofs*

By O. GRUENWALD and W. A. RENNER, ERIC C. MOLKE, ROBERT ZABOROWSKI and AUTHOR

By O. GRUENWALD and W. A. RENNER†

Whitney's paper on the cost of long-span concrete shell roofs is timely and serves well to acquaint the engineering profession with the cost of this type of construction. Exception however is taken by the writers to several of the statements made and to some of the calculations presented in the author's evaluation of the importance of the location of the shell with respect to the axis of the arch ribs.

Whitney states, "Since the moments due to volume change and the abutment rotation may be almost as great as the live load moments on a structure of this type, it is evident that the location of the shell with respect to the rib can greatly affect the economy of the construction." He then proceeds to evaluate this effect on the arch ribs by analyzing the crown section of a 220-ft span arch, both for the case where the shell is located at rib mid-height and where the shell is attached to the top of the rib. Applying live load and volume change moments and thrusts consistent with the moment of inertia of the two cases he concludes that the section with the shell at the top of the rib has insufficient capacity and must therefore be increased in size. A simple calculation, however, shows that these results are erroneous.

Using the same moments, normal forces and section properties as given by the author and computing the stresses using the formula $P/A \pm Mc/I$ for an uncracked section it is found that the section with the shell attached to the top of the rib need not be increased in size. Actually, the concrete compressive stresses are lower and the buckling safety is 100 percent greater than when the shell is attached at mid-height of rib. Pursuing the matter further, calculations were also made for a reduced section with the shell at the top. Here too, the compressive stresses were found to be lower and

*ACI JOURNAL, June 1950, *Proc.* V. 46, p. 765. Disc. 46-56 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Structural Engineers, Roberts and Schaefer Co., Chicago, Ill.

the buckling safety higher than for the case with the shell at mid-height of rib—a direct contradiction to Whitney's conclusions. (See Table A).

Although the compressive stresses for case 3 are slightly lower than those of case 1, the allowable stress, based on the reduction formula of the ACI Code, is exceeded slightly in case 3. The minimum steel requirements, however, reduce the compressive stress to a value within the allowable.

Having demonstrated the inaccuracy of the author's conclusion that for a given set of moments and normal forces, shifting the shell from the middle of the rib to the top or bottom of the rib could make the section inadequate, the writers do not believe it justified to assume that an investigation of the crown of the arch alone can determine the most economic cross section. Not only do the maximum moments vary in sign and magnitude along the arch axis, but also the relative importance of the volume change moments varies. In the lower quarters of the arch the volume change moments are only a small percentage of the total design moments. For example, the section of case 4, Table A, would be inadequate in the region of maximum negative moment without excessive reinforcing.

Comparisons were made by preparing complete designs of roof systems with shells located at the center of the rib and at the bottom of the rib. For identical loading conditions these designs indicate less than 1 percent difference in cost rather than 30 percent as stated by the author. Such comparisons are particularly influenced by the cost appraisal of handling the scaffolding. In the one case the decentering and moving operation is simple; in the case with the shell at the center the projection of the arch rib into the scaffolding causes a complication which in turn produces a corresponding cost penalty.

A further point to be gained is the increased buckling safety which is developed by placing the shell at the bottom, or top, of the arch rib. If the designer wishes, he may take advantage of this by reducing the cross section and still maintain a given safety to buckling. If not, for the same amount of material, the buckling safety is increased and the decentering time is lessened when the shell is attached to the bottom of the rib.

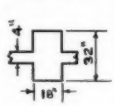
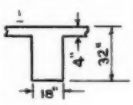
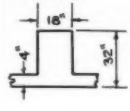
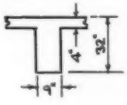
By ERIC C. MOLKE*

Whitney has made a great contribution to the art of designing long-span concrete shell roofs by shaping shell and arches to the funicular line for dead load. Attention should be called to the force action caused by the funicular shape taking place in such a shell.

In line with the theory of least work, an arch will carry as great a part as possible of the loads by direct forces and will only act in bending for that part of the load which has caused the funicular line to deviate from the given arch shape. In other words, if the arch is shaped to the funicular curve, there will only be a direct compressive action and no moment stresses will

*Consulting Structural Engineer, Highland Park, Ill.

TABLE A—EFFECT OF LOCATION OF SHELL ON STRESSES AND BUCKLING SAFETY OF ARCH RIB

	Case 1	Case 2	Case 3	Case 4
				
Moment of inertia of crown section, in.^{4}	58,000	116,300	116,300	69,700
Moment due to live load, in.-lb	1,710,000	1,710,000	1,710,000	1,710,000
Moment due to volume change, in.-lb	1,026,000	2,042,000	2,042,000	1,230,000
Total moment, in.-lb	2,736,000	3,752,000	3,752,000	2,940,000
Max. horizontal thrust, lb	447,120	450,000	450,000	408,000
Concrete stress at top of arch, psi	- 1096	- 628	- 1091	- 686
Concrete stress at bottom of arch, psi	413	404	- 59	663
Total tension force in concrete to be taken by reinf., lb	32,500 (min. reinf.- 2.9 sq in.)	44,500 (min. reinf.- 2.9 sq in.)	0 (min. reinf.- 2.9 sq in.)	46,000 (min. reinf.- 2.3 sq in.)
Relative buckling safety of arch - proportional to I (Dischinger's method)	8.7	17.4	17.4	10.5

occur. It follows that in Whitney's construction, the live load will be the principal cause for the moments in the arches.

A concrete shell roof really consists of two different arch systems, each carrying part of the load in the direction of the arch span. The first arch system consists of the thin shell arch on which, due to its large area, most of the loads are applied. The second system consists of the arch ribs proper. Both systems are abutting against the same piers. Since the shell slab is not supported continuously on the piers, shearing stresses are caused in the shell near the abutments.

As reasoned above, only compressive action without bending will occur in both systems for dead load. The dead loads are caused principally by the weight of the concrete. Consequently the unit compressive stresses in both

systems will be approximately the same, the loads being proportional to the cross sections. One can therefore conclude that both the shell arch, as well as the arch ribs, will tend to settle an equal distance, side by side, due to rib shortening. Consequently there should be no strains caused by dead load along the edge of the arch ribs where the two systems are connected.

The same reasoning can be applied for live loads, if like the dead loads, they are evenly distributed over the shell so that their funicular line will coincide approximately with the arch line. When discontinuous live loads are applied however, a substantial interaction between shell arch and arch ribs will occur, causing strains at their connection.

For example, for the case of a one-sided live load the funicular curve will deviate considerably from the arch line. The system of the shell arch cannot carry such loads because it is too thin to take upon itself the resulting bending moments. Since the shell arch is well suited to resist all such loads for which the funicular curve coincides with its shape, it will only unload to the arch ribs that part of the loads which caused the deviation of the funicular curve from the arch line.

A corrective load distribution between shell arch and arch ribs will therefore take place, causing principally membrane shears, cross bending moments and bending moments in the direction of the arching, all extending over the whole shell area. This corrective load distribution will be of varying degree throughout and especially along the connection with the arch ribs. In overloaded zones, where the funicular line is higher, part of the shell loads will be transferred to the ribs. At the unloaded side, where the funicular line is lower, part of the weight of the arch ribs will actually be shifted toward the shell. The final effect of this corrective transfer of loads is to force the resultant funicular curve into the shape of the shell slab.

The continuous cross bending moments in the shell slab (Fig. 1), which apply only to a full circular pipe under water pressure, will therefore not apply for dead load and only in part for live loads.

It is interesting to note that Whitney's choice of the funicular curve for the arches results in practically eliminating the three-dimensional so-called "shell action" occuring on cylindrical structures of any other shape. As has been shown, the two arch systems operate parallel and independently for the major part of the loads. The shell arch carries its load downward along vertical planes, practically right to the abutments, without unloading itself onto the arch ribs.

On most earlier shell structures, the cross-sectional line of the arches was located above the funicular curve. This induced a longitudinal load transfer from the shell to the arch ribs, by three-dimensional shears in the plane of the shell, for that part of the load which caused the funicular line to lie above. The resulting stress distribution was illustrated in Fig. 19, p. 678, May-June 1938 ACI JOURNAL.*

*Molke, Eric C. and Kalinka, J. E., "Principles of Concrete Shell Dome Design," ACI JOURNAL, May-June 1938, *Proc.* V. 34, pp. 649-708.

The longitudinal shifting of loads to the ribs by this so-called shell action, while it reduces the stresses in the shell slab, must on the other hand cause higher stresses in the arch ribs. Whitney's choice of the funicular shape is therefore greatly justified on long spans where economy in the arch design is much more important than economy in the design of the shell slabs.

By ROBERT ZABOROWSKI*

Having been associated with Roberts and Schaefer Co. for many of the years since their introduction of shell structures to the United States in 1932, the writer finds Whitney's remarks of considerable interest. It is always stimulating to get new points of view. There are, however, a number of points made by the author that bear further comment, matters dealing both with design and economy.

In introducing a new medium of construction we found that engineers regarded with skepticism, and justifiably, the alleged economy of the thin shell-arch system when compared to other established forms of construction. The economic possibilities of shell construction were forcefully demonstrated in 1941 with the Columbus warehouse program of the Army Quartermaster Corps. A thin shell concrete structure was bid as an alternate to a temporary, structural steel, wood-deck building. Ten bids were received; the award was made for concrete with a 4 cent per sq ft saving over the lowest figure for temporary construction. In fact, of the more than ten million sq ft of shell construction we have engineered, representing buildings from small warehouses to the largest hangars, almost every single project depended on prior economic justification before the final decision was made to use a concrete shell structure.

Coincident with Whitney's interest in shell structures we began to hear a great deal about the significance of placing the shell at the middle of the supporting rib. We are indeed in an age of specialization! Whitney has estimated that by this simple expedient the cost of shell structures can be reduced by as much as 30 percent.

Whitney reports the cost of the American Airlines hangars, built in 1947, as $12.90 per sq ft. The Rapid City, S. D., hangar which we designed for the Corps of Engineers in 1947 has a 340-ft clear span, approximately 80 ft greater than the American Airlines hangars. The low bid on the Rapid City hangar was $1,343,000, or $12.16 per sq ft complete.

The two projects provide a means to check the author's assertions. According to his price comparisons, connecting the shell at the bottom of the arch imposes a 30 percent unit price increase over the mid-arch connection. Also, his graphs relating cost with transverse span indicate about a 15 percent unit price increase in going from the American Airlines hangar span to the 80 ft larger Rapid City span. These two items together imply that the unit price of the Rapid City hangar should have been considerably higher than the American Airlines hangar. However, this theory is not supported by

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fact, as the Rapid City hangar unit price was less than that of American Airlines hangar.

If the mid-arch position does not justify itself in economy, we might examine its merits from a design point of view. Whitney states an important consideration in the design of wide span shell arch structures—buckling stability of the roof structure. The stability of any arch rib is directly proportional to the effective moment of inertia. Generally speaking, for large structures, the effective moment of inertia of an arch is approximately 80 percent greater with the shell at the bottom or at the top than with the shell at the center. Thus, for the same amount of material a safer structure is achieved by placing the shell at the top or at the bottom.

The author's illustrations of the effect of moment inertia on bending moments produced by shrinkage and temperature change are accurate but misleading because these moments alone do not control the design. They are only a small fraction of the governing bending moments which are produced by full-intensity, partial live loads.

In 1936 the Hershey Chocolate Co. retained our firm to design a large monolithic concrete shell type arena. Aside from being the first of its type in the country, it is one of the few examples of such construction that was built with a minimum of skilled labor; a major portion of the construction personnel was drawn from Hershey's regular employees. One of the structural characteristics of this job is the connection of shell to the top of the arch rib for a major part of the span. Toward the springing lines the shell crosses the face of the rib to the bottom. The design force felt that it had achieved a particularly economic structure. In the field, however, we quickly learned that the projection of the rib into the falsework was a costly complication. Our own studies indicated that both the savings in materials effected by this design and the savings in the roofing operation made possible by eliminating flashing at ribs were more than offset by the increased cost of form handling.

Frankly, no real merit is seen in the mid-roof position of the shell except that of novelty. Constructionwise, it is more costly; designwise, it reduces structural safety for the same amount of material when compared to the top or bottom rib position.

One phase of our established research program is the current testing and analytic project at the Fritz Engineering Laboratory at Lehigh University. There is in progress a comprehensive investigation of the structural characteristics of thin shells and the results of this work will be published as they become available.

AUTHOR'S CLOSURE

Molke presents a discerning description of the general behavior of shell structures of the type pictured in the paper. As he correctly states, the shaping of the arch-rib to the funicular line for dead load eliminates most of the rib moments due to dead load and uniform live load. It is then necessary to

provide only sufficient flexural strength to resist moments caused by partial live load, volume changes, abutment motion and deflections. Provision of optimum flexibility reduces the total of these moments to a minimum and results in greatest economy.

The effectiveness of these principles is best proven by contrasting the appearance and performance of the roof of the American Airlines hangar with all long-span arched shell structures built before it was constructed.

While the writer does not wish to detract from the credit due to their firm for pioneering in the use of Zeiss-Dywidag shell structures in the United States, he does find it necessary to point out that the discussions of Zaborowski, Gruenwald and Renner are unrealistic and indicate the need of design standards for such structures.

Zaborowski has chosen to compare the actual cost of the American Airlines hangar with the bid price on the Rapid City hangar but failed to point out that the actual cost per square foot of the Rapid City hangar was much higher than the American Airlines hangar. He misrepresented the graph of Fig. 9 as indicating an increase of 15 percent in unit cost due to increase in span from that of the American Airlines hangar to that at Rapid City. The graph indicates an increase of about 70 cents per sq ft which is much less than 15 percent of the unit cost of the hangar. The American Airlines hangar consisted of two short barrels instead of one long one thereby adding to the cost because of the two extra end walls and the long lateral travel of the falsework. It is the writer's opinion that the two jobs were different in so many respects that comparison of costs is meaningless.

Gruenwald and Renner's Table A illustrates that there is little agreement between their design concepts and those of the writer. Their calculations appear academic in the extreme.

In slender arches of this type, the thrust eccentricity under critical loads is so great that it is not permissible to assume an uncracked section. All sections of the rib are subjected to reversal of bending under partial loading; consequently the shell may be on the tension side of the rib if it is placed either at the top or the bottom and it is necessary to design the rib as a rectangular section as in the case with the shell at the center of the rib. For these reasons, the calculated stresses tabulated in Table A appear to be meaningless.

The tabulated buckling safety figures are also without significance. They are evidently calculated for some one particular loading under the assumption that the material has no elastic limit. Since the factor of safety against steel failure may be only about 2 to 2½, a theoretical buckling factor of 17.4 can be no better than 8.7 and the consequent stiffness simply adds to the moments acting on the rib.

It is the writer's practice to protect the arch from buckling by calculating the moments due to deflections caused by a multiple of the critical live load for each section in combination with dead load and all other effects. Because the deflection moment is caused by the *total* thrust operating at an eccentricity

due to the deflection, it increases much more rapidly than the live load increment and it cannot be calculated by multiplying the conditions under design load by a factor of safety. Instability in the arch would primarily be caused by an increase in bending moment, not in thrust as assumed in Gruenwald and Renner's calculation.

Deflection stresses increase with flexibility but those due to volume changes and abutment movements increase with stiffness. The importance of obtaining that degree of flexibility which will minimize the sum of those effects cannot be overemphasized. Over-stiff shell structures are subjected to high stress because of their stiffness which has caused serious cracking and actual failures. The calculated moments due to volume changes and foundation displacement of the American Airlines hangar, the Onondaga County War Memorial and the Alabama Livestock Coliseum pictured in this paper averaged 91 percent of the live load moment at the crown and 43 percent at the springing. Doubling the moment of inertia would have increased these moments to more than 182 percent and 86 percent of the live load moment because stiffness of the strengthened rib would have increased accordingly. This increase in moments and thrusts due to stiffness increases the cost of the abutments and footings as well as the ribs.

In spite of the fact that the rib must be designed as a rectangular section regardless of the location of the shell, the shell is severely loaded if placed at either the top or the bottom of the rib because of its action as a flange. In developing tensile or compressive stresses as a flange, due to curvature of the shell, an unbalanced radial force is developed which produces bending stresses in the shell similar to those caused by live load. For example, if the axial compression in the flange is 400 psi and the shell is 4 in. thick, the radial component per foot for a shell radius of 200 ft is approximated as $\frac{400 \times 4 \times 12}{200} = 96$ psf. This is greater than the usual dead and live load and must be considered in the design of the shell.

The cost of the falsework and forms as affected by position of the rib can easily be estimated and should not be much different for properly proportioned ribs whether the shell is at the bottom or mid-height. The cost of forming the lower projection of the rib is incurred only once in the movable falsework. That of lowering the falsework and placing forms for the upper projection above the shell occurs for each pour. The American Airlines hangar, for instance, had more than a mile of upper rib to be formed and poured. Extra height of upper rib form is expensive. With ribs properly proportioned, the projection below the shell will normally be only 1 or 2 ft and the extra cost of lowering the falsework is not serious. In the case of the Hershey Arena cited by Zaborowski, the maximum vertical projection of the rib below the shell was about 15 ft and no doubt the operation was costly.

Alertness to changes in mix requirements and job conditions are essential in the

Inspection of Ready-Mixed Concrete*

By E. L. HOWARD†

SYNOPSIS

The variety of mixes produced in a ready-mixed concrete plant complicate the work of the inspector in varying the mix design to suit the consumer's desires. Materials handling methods, changes in brands of cements and the means provided for measuring admixtures add to the difficulties. Aids to overcome these difficulties are discussed and a meter for measuring consistency of concrete is described.

MATERIALS CONTROL

There are many inspection and control problems peculiar to ready-mix concrete operations. A contributing factor to some of these problems is the great variety of concrete produced. It is not unusual to use as many as 48 different mixes in the course of a week. When changes in the characteristics of cement, aggregates, or both, necessitate mix adjustments, the inspection staff has dozens of mixes to change in a hurry. It is important, therefore, that suitable testing and controls be exercised at the source of the materials.

In the case of aggregates, assuming that materials are tested and controlled at the mill to meet the specifications, the inspector still has the problem of determining the changes batch plant handling makes in them. For example, tests of coarse aggregate at one batch plant showed 10 percent breakage due to methods of handling. Another plant had little breakage but considerable separation of all sizes of aggregates in the bins. The inspector will find that each plant will have its own peculiar materials problem and he must consider its effect on his concrete design.

The inspector must know the difference between the several brands of cements he uses and design his mixes with these differences in mind.

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Two tables have been organized from our own research to make concrete mix design more nearly accurate using the Type I cements available to us. Table 1 shows the comparison of several brands of cements to the average. Table 2 indicates the W/C ratio needed for stated compressive strengths using each of several cements.

After the mix is designed or adjusted, the plant operator is provided with a card showing the batch weights for several volumes.

Many mixes are furnished by the customer and almost always these batch weights are calculated to the nearest pound. For example, a

TABLE 1—RELATIVE COMPRESSIVE STRENGTHS OF CONCRETE MADE USING SEVERAL BRANDS OF CEMENT*

Brand of cement	Percentages of average 28-day compressive strengths, psi		
	Sacks cement to cu yd of concrete		
	4	5	6
Untreated concrete, average psi	1340	2653	3700
A	109	110	107
B	115	100	97
C	86	101	94
E	79	102	95
F	126	108	103
Air-entraining concrete, average psi	1980	2950	3690
A	121	108	110
B	91	74	101
C	101	123	112
E	107	119	97
F	79	76	80

*¾-in. maximum size aggregate, 4 to 5-in. slump.

TABLE 2—COMPRESSIVE STRENGTHS FOR VARIOUS WATER-CEMENT RATIOS USING SEVERAL BRANDS OF CEMENTS

Net water-cement ratio by weight						Probable strength 28-day, psi
Cement brand						
A	B	C	E	F	G	
0.65	0.55	0.55	0.51	0.53	0.51	4000
0.71	0.62	0.60	0.57	0.63	0.57	3500
0.77	0.68	0.66	0.63	0.74	0.63	3000
0.83	0.74	0.73	0.68	0.83	0.68	2500
0.89	0.85	0.79	0.76	0.91	0.76	2000
0.96	1.13	0.86	0.88	1.12	0.88	1500

mix might be in lb moist weight, 1½-in. gravel—1184, ¾-in. gravel—746, coarse sand—1151, fine sand—238, cement—541, and admixture—2.

In a batch plant, scales usually are marked in increments of 10 or 20 lb. The operator must decide where to set the scale for this batch. Shall he make the 1½—1180 or guess the point 1184 should be? In either event he must guess on every weight. To relieve the operator of these decisions our mix designs are computed to the accuracy of the batch plant scales.

The inspector finds plenty of headaches in putting admixtures into concrete. The plant may have metering devices for some brands but others are measured in something resembling an old coffee tin. Many strange and startling things happen to concrete when these less than accurate methods are used.

When admixtures are improperly measured, don't be surprised if the concrete does not set at all or perhaps sets so fast you can't place it. Perhaps the yield is long or short or maybe cracks and crazing will appear in a finished slab.

The inspector must carefully inspect the batching facilities and recommend the measuring device best suited for the admixture used. Rather than sacrifice the concrete quality it is better to substitute an admixture that can be easily and accurately measured.

When the sales department sells concrete to customers each with his own set of specifications, the inspector prays for a miracle. More often than not the plant can bunker materials under only one set of specifications. It is the multiplicity of specifications that causes the ready-mixed concrete operator no end of problems.

The man placing the concrete wants uniformity above all else. He measures this uniformity by what he calls slump. The inspector soon learns that Pete Doaks wants 6-in. slump concrete but Pete calls it 4 in. Even proof of his poor guess won't change Pete's eyeball test results.

The inspector finds that aggregate moisture changes, mixing time and speeds, differences in transit trucks, and many other things effect the uniformity of the concrete. The big problem is to get notice of these changes to the plant operator quickly. Jobs are scattered and sometimes miles from the plant.

INSPECTION AIDS

Sand moisture test

We have developed a quick and easy method of estimating moisture changes in the sand. The method is subject to errors but the careful and experienced operator may get accurate results. A 250 ml graduated

mixing cylinder with stopper is used for the test and except for a graph (Fig. 1) no other equipment is needed. Into 110 ml of water approximately 150 grams of moist sand are introduced. Readings are made of the volume of settled sand and height of water column. Percent of moisture can be determined from values obtained from the graph.

Consistency meter

There has been a need for fast and accurate measure of consistency in concrete. Last year J. W. Kelly loaned the author a device referred to here as the Kelly Consistency Meter (Fig. 2). Mr. Kelly had a goodly number of laboratory tests to show the efficiency of the device. He describes the meter or ball test as follows:

"The test eventually adopted consists in observing the penetration of a 30-lb plunger having a 6-in. hemispherical tip into the leveled-off surface of the concrete. The penetration in inches is read on the graduated handle of the plunger, at the top of a sliding stirrup with two feet each 1 in. square, the centers of which feet rest on the con-

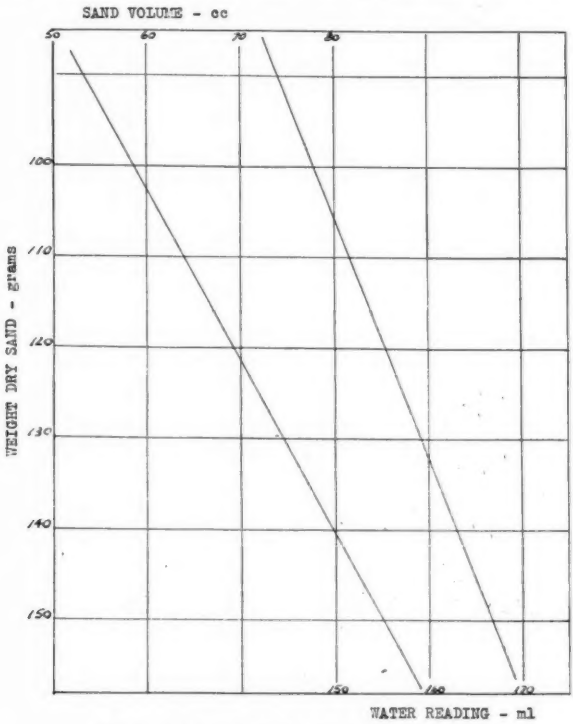

Fig. 1—Moisture chart for Centerville mill sand

Fig. 2—Kelly consistency meter

crete at a distance of 5½ in. from the axis of the hemispherical tip. The penetration is read to tenths of an inch, usually as the average of three readings. The concrete may be in a container, in a wheelbarrow or cart, or in the forms, so long as the depth is at least 6 in. and the least horizontal dimension is 12 in.

The original form of the device was a 6-in. cast-iron ball. A more economical form to manufacture was machined from a 6-in. steel cylinder, and the length of the cylindrical portion was made such that the plunger with handle weighs exactly 30 lb. Observations to date indicate no significant effect of the difference in shape (as between ball and cylinder) of the upper portion of the plunger, which comes into play only when the concrete is of wet consistency."

We took the meter into the field and made consistency measurements at the truck, in hoppers, buggies, forms, slabs, and on pavements ahead and behind finishing machines. The job foremen were happily surprised at the simplicity of operation. In a few seconds the consistency can be measured. A load of concrete can be tested from plant to forms through every phase of the transportation without delay. We found that the uniformity of concrete as delivered to the job was much better than the uniformity at the point of placement. The trip from truck to forms via chutes, hoppers or buggies can cause a great amount of mortar-aggregate separation.

Mr. Kelly had found that a meter reading could be translated into terms of slump by using the factor 2. Thus, a meter penetration of 1.5 would be equivalent to approximately a 3-in. slump. Our field experience confirms his findings. Using the meter has convinced us that great accuracy in consistency measurement is obtained and when the slump does not agree with the Kelly meter we rather think the fallacy lies with the slump test. We made slump tests and Kelly meter readings

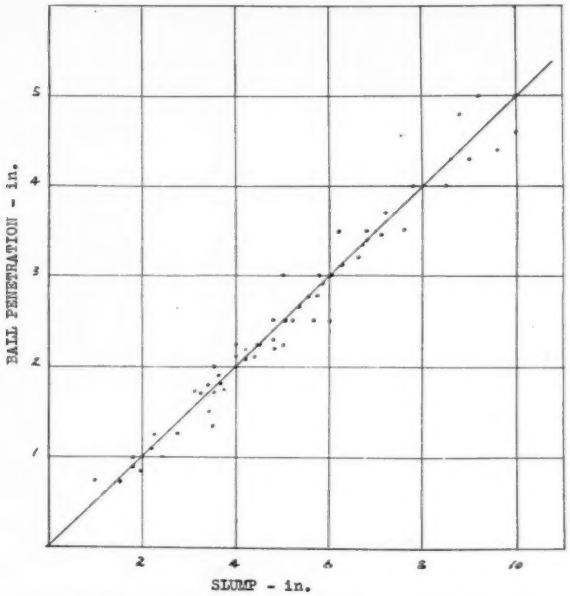

Fig. 3—Comparative consistency measurements of fresh concrete

of concrete of all types produced in our ready-mix plants. Results of these tests are plotted in Fig. 3.

SUMMARY

The great variety of concrete jobs served by ready-mix plants with the attendant problems of changing cement and aggregate requirements, batch scale changes, varied distances and haul times to jobs, and inconveniences of on the job sampling and testing makes the inspector's work day one of interesting adventure.

What an inspector should be and do.

Summary of Inspection Practice*

By R. B. YOUNG† and W. SCHNARR‡

SYNOPSIS

The authors summarize the seven papers on inspection given at the 46th annual convention and published one a month in the ACI JOURNAL from December, 1949, to June, 1950.

There has probably been no better discussion of the problems of concrete inspection than has been presented in the series of papers on inspection concluded by that of Mr. Howard, p. 777. The several authors differ in the emphasis they place on various phases of the inspector's functions, depending on their individual experience and the field of construction in which they are active. But in spite of these natural differences it is interesting to note their essential agreement on such points as the qualifications of an inspector, his basic duties and the responsibilities of his employer.

It is agreed by all that to be a successful inspector, a man must be of good character, like his work, have a large measure of common sense and tact, and command the respect of his associates. He should know his job thoroughly, be firm in carrying out his duties, and enforce the specifications impartially in all essential particulars. But he should have an appreciation of the contractor's problems and co-operate with, rather than harass him. This may seem like the specification for a superman but it has been the authors' experience that by careful selection and training, men closely approaching these requirements may be had.

No matter how good he is, the inspector cannot fulfill his duties satisfactorily without the support of his employers; all the authors agree on this point. To this end the inspector should be instructed specifically as to his responsibilities and authority so that there can be

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no misunderstanding by him as to what questions he can decide and what have to be referred to his superiors. Within this authority, his decisions should be upheld unless he has made a serious error in judgment. There is nothing so demoralizing to an inspector as to have his decisions reversed and a few experiences of this kind will destroy utterly his usefulness and deprive him of all authority and standing on the job. Again it is poor policy for an owner to listen to complaints or criticisms regarding his staff in a way that gives the complaintant reason to feel that he may get concessions. The contractor should be made to work out his differences directly with the inspector, and most contractors would rather do so, but they would be less than human if they didn't take advantage of the owner who by a little pressure can be persuaded to interfere with his staff in the carrying out of its duties.

Another important point repeatedly brought out by the several authors is the responsibility of the owner or employer to supply the inspector with all necessary information about the work for which he is responsible. This means an adequate and enforceable specification describing the work to be done and the standards of quality to be met, complete and detailed drawings and any other pertinent information that he needs to properly carry out his duties. An inspector is not psychic, he cannot know what the contract requires if he hasn't these documents and the more precisely they set forth the requirements that govern the work, the better he can do his job. Likewise the inspector should be kept informed of all changes that have been agreed upon between the owner and contractor, either in the drawings, specifications or contract, and this information should come to him from the owner and not have to be learned through the contractor.

To enforce the specifications, the inspector must be thoroughly familiar with their requirements. It is surprising how often neither the inspector or the contractor's representative on the job really know the contents of the specifications governing the work they are doing. Many of the arguments that arise on the job would have never occurred if they had been familiar with its details. But whether the contractor knows or not, the inspector can save a lot of trouble for himself and his employer if he has learned the contents of every clause in the specification. And more trouble would be saved if the inspector and contractor would adopt the practice of discussing each part of the work before it got underway with a view to arriving at a mutual understanding of how the work was to be done and of what the owner is going to require. This is the time to iron out differences of opinion rather than after the work in question is started.

It is evident from the different inspection papers that while the inspector's basic responsibilities are substantially the same whatever

the work, his actual duties vary greatly with its type and size. The inspector on a building job has to deal with many trades and the concrete may form only a minor part of the structure, or he may be employed by a ready-mix operator where his responsibility for the concrete ends with its delivery to the customer. Again, the inspection of concrete for highways differs in many details from either the ready-mix or building job and both from a dam. And the size of the job enters into the problem. Where the job is small, one man may be able to supervise all concreting operations; where the job is large, it may require a dozen or more. The single man, to do his work properly, needs to have a working knowledge of all phases of concreting, *i.e.*, materials, mixing, placement, finishing and curing. On the large job, where inspection is performed by a group it will be headed by a man who needs to be both an executive and an expert concrete technologist but no such requirements are necessary for the other members of the group for they are under close supervision and usually are assigned to deal with only one phase of the work such as mixing or placement, hence their knowledge of concrete can be more limited as long as they have the proper personal qualifications.

Although the several papers agree in the main as to the inspector's basic duties, they differ substantially in the importance they attach to each. This has its genesis largely in the different types of structures with which their authors deal. For architectural concrete, appearance is a major consideration and perfection in finish and details is strived for. Consequently, Dunlap* places principal emphasis on forms and placing and to a slightly less extent on curing. On the other hand, in dealing with the inspection of massive concrete structures, Tuthill's conception† of inspection is a balanced control of all concreting operations from the processing of the cement and aggregates to the final curing and protection of the concrete. Stadtfeld,‡ who also deals with monumental structures, places great importance on the mill inspection of cement. He holds to the view that disintegration of concrete is largely the fault of the cement and gives in some detail the methods his organization follows in insuring its quality. Russell,§ working in the highway field, stresses the importance of controlling the production of concrete by close supervision of materials, proportioning and air content. Howard's inspection problems¶ are also those of controlling

*Dunlap, Leonard E., "Inspection of Building Construction," ACI JOURNAL, Apr. 1950, *Proc.* V. 46, pp. 609-612.

†Tuthill, Lewis H., "Inspection of Mass and Related Concrete Construction," ACI JOURNAL, Jan. 1950, *Proc.* V. 46, pp. 349-360.

‡Stadtfeld, N. T. F., "Inspection and Testing of Materials," ACI JOURNAL, Dec. 1949, *Proc.* V. 46, pp. 237-248.

§Russell, H. W., "Inspection and Control of Concrete for Highway and Bridge Construction," ACI JOURNAL, Feb. 1950, *Proc.* V. 46, pp. 437-444.

¶Howard, E. L., "Inspection of Ready-Mixed Concrete," ACI JOURNAL, June 1950, *Proc.* V. 46, pp. 777-780.

materials and proportions, but are complicated by the many different concrete mixtures produced in a ready-mix plant. He also points out the importance of control of the aggregates at their sources.

Several of the papers mention testing as a part of inspection: It is a very necessary part. Whether it is done by the owner's organization or by a commercial laboratory is not important so long as the group doing it is competent. In urban areas it is an increasingly common practice for commercial laboratories to furnish inspection services as well as doing testing. This has many advantages, for assuming that the staff of the commercial laboratory has the necessary know-how, it greatly simplifies the problem of finding competent inspectors and it provides for their supervision by specialists. But if a commercial laboratory is employed, the work of their men should be co-ordinated with the job organization of the owner, their authority and responsibility should be clearly defined and their competency ascertained. Further, as pointed out by Dunlap, they should be paid by the owner, not the contractor. This is fundamental, for otherwise, how can they be independent? It seems naive to the authors to ask an inspector to impartially represent the owner when his salary is paid either directly or indirectly by the contractor he is asked to check. And it should be added that the commercial laboratory should be chosen for their competence and not because they offer to do the work for less.

Inspection often is held in low regard because of the type of men employed. This stems from several causes, the appointment of men who have no other qualification than need of a job, the far too common view that inspection is a job for a junior as a place to gain experience and finally the reluctance of owners to pay salaries that will attract experienced men. As suggested by Andrews and Gray,* the inspector should be selected "only after careful consideration of his experience in doing the type of work involved" and we would add that he should be paid on the same level as his counterpart in the contractor's organization.

Every engineer and owner responsible for inspection should study the paper by Andrews and Gray. An appreciation of the contractor's viewpoint and his problems, the difference in his objectives and those of the owner and the degree to which human relationships affect the smooth functioning of a job would do much to reduce conflicts between them. And we would suggest that the paper be studied by contractors as well for too few contractors have as good a grasp of the problem inherent in inspection as Messrs. Andrews and Gray.

*Andrews, Donald C. and Gray, Nomer, "The Contractor's Viewpoint of Inspection," ACI JOURNAL, Mar. 1950, *Proc.* V. 46, pp. 509-512.

A survey of present practice for joints in concrete pavements.

Design and Construction of Joints in Concrete Pavements*

By WILLIAM VAN BREEMEN† and E. A. FINNEY‡

With a supplement on

Structural Design of Joints for Airport Pavements

By THOMAS B. PRINGLE§

SYNOPSIS

The basic fundamental conditions involved in the design and construction of joints are discussed. Various types of joints and load transfer devices are described, as well as various forms of structural failure of joints. Recommendations are given for the design, fabrication and installation of load-transfer devices, the construction and finishing of the concrete surrounding the joint assembly and the sealing of joints. A supplement discusses joints for heavy duty airport pavements.

INTRODUCTION

As a part of the work of ACI Committee 325, Subcommittee 3 was established and assigned the subject of structural design of joints. This paper has been prepared in cooperation with the members of that subcommittee and has for its purpose a review of jointing practice in highway pavements as it exists today. In general, it includes the definition, description and function of joints, joint design, load-transfer devices and their function, forms of joint failure and trends in joint design.

From all viewpoints, an ideal concrete pavement surface would be one consisting of a continuous ribbon of concrete of uniform construction with no breaks in its continuity. Unfortunately, however, such a

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pavement cannot be economically achieved under present-day practice although studies are being conducted in an attempt to accomplish this, in effect, through the use of continuous, heavy reinforcement.

Because of its inherent weakness in tension, concrete is highly susceptible to cracking under tensile stresses, particularly those induced by volume changes due to temperature fluctuation, autogenous shrinkage of the concrete at early age, or other causes. Even in the "continuously reinforced" design the concrete itself is not continuous. When transverse joints are not included in the design they will, in effect, be supplied in the form of transverse cracks that will sooner or later occur. The nearest approach to obtaining an ideal pavement comparatively free from transverse cracks has been accomplished by dividing the pavement into a series of slabs by the introduction of joints of one kind or another, the slabs to be as long as possible consistent with practical design requirements and within economic limitations.

Depending upon their design, the function of such joints is to maintain within safe limits the stresses caused by expansion, contraction and warping of the concrete. Transverse joints are thus functionally classified as "expansion," "contraction," and "warping" or "hinge" joints. Longitudinal joints, as the name implies, are placed lengthwise in the pavement and act essentially as continuous hinge or contraction joints, depending on the method of construction. If the longitudinal joints contain a filler they may also act as expansion joints, as may be the case in wide sections of airport pavement.

It may be said that every joint is potentially a source of structural weakness contributing to the ultimate deterioration of the pavement. They may also limit the load-carrying capacity of the entire structure. Since it is possible to design a concrete pavement slab strong enough to withstand combined warping, load and friction stresses for any predictable traffic condition and subgrade support, it would appear to be good engineering practice to reduce the number of joints to an absolute minimum, consistent with sound design principles.

The structural inadequacy of transverse joints has been fully appreciated by highway engineers for many years and many designs have been advanced as a solution to the problem. This is reflected in the voluminous literature on the subject. However, up to the present, the joint problem has not been fully solved, and perhaps it never will be unless a conscientious and concerted effort is made by those in authority to remedy the following serious deficiencies which, in general, have exercised a profound influence on the design, construction, and performance of joints during past years:

1. The permission of wide latitude in the interpretation of pavement specifications.

2. The prevalence of considerable lenity in the inspection and control of construction processes.

3. Careless practices associated with the initial sealing of joints and the subsequent maintenance thereof.

4. The lack of a suitable joint filler and sealing compound.

5. The inability of present subbase construction practices to offer uniform and continuous support for the slabs.

6. The pursuance of a short-sighted policy with respect to the cost of joint assemblies and load-transfer devices without full appreciation of their relative mechanical efficiency or their effect upon the ultimate service life of the structure.

7. The comparatively slow progress being made in the development of basic information and sound design theories in relation to joints.

8. The general practice of allowing steel fabricators or others, rather than state and federal agencies, to take the initiative in joint design.

After more than a half century of experience in the design and construction of concrete pavements, there is still no generally uniform practice for either the use and spacing of joints or the design and construction of the joints themselves. It is doubtful, however, whether uniform practice may ever be achieved, or in fact even be desirable, in view of the many variations in climatic, traffic and soil conditions in the United States. We nevertheless firmly believe that present jointing practice can be materially improved by (1) correcting, so far as is practical, the deficiencies enumerated above and (2) coming to a mutual understanding on certain fundamentals of joint practice common to all—subgrade support, workmanship and specifications for the evaluation, spacing and placing of load-transfer devices in joints.

FUNCTION, DESCRIPTION AND DEFINITION OF TRANSVERSE JOINTS

In general, a transverse joint may be defined as an artificially created plane of interruption in the continuity of the pavement. The joints divide the pavement into a series of slabs of predetermined length. Since there are several kinds or types of transverse joints, some discussion should be devoted to their basic features and specific functions.

Because of the general looseness in the use of descriptive terms for the various types of joints it is believed desirable at this point to assign definite terms to each kind and type to avoid ambiguity and misunderstanding. For the sake of clarity and mutual understanding, definitions and functions of the respective joints are presented below. It is hoped that the terminology employed will eventually meet with general acceptance.

It may be stated that all types of joints can be placed in one of three basic categories, namely, "expansion," "contraction" and "hinge." Examples of joints falling in each of these three categories are presented in Fig. 1.

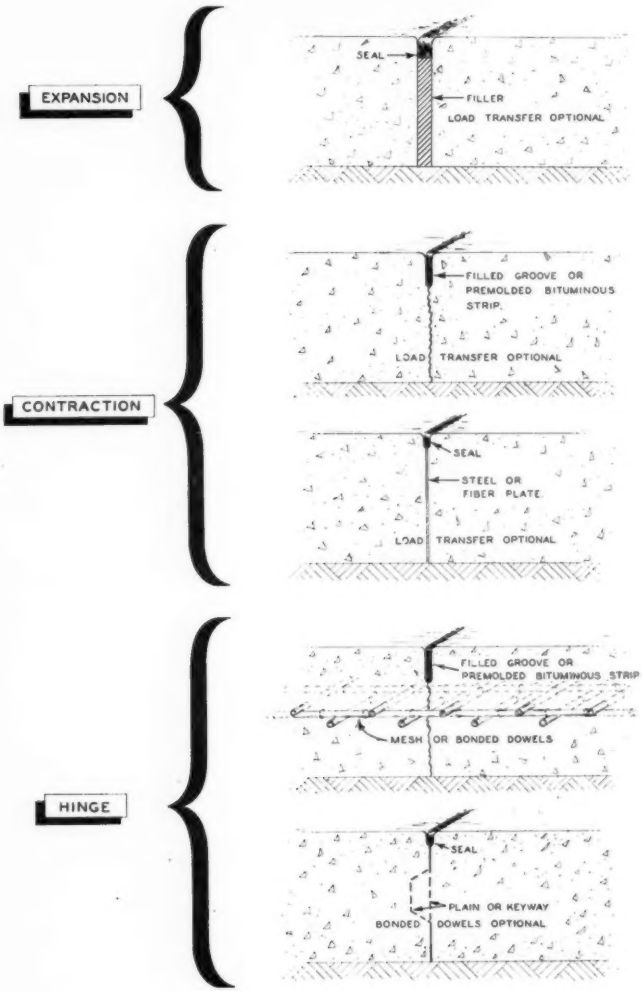


Fig. 1—Three basic types of joints

Expansion joints

As the term implies, the primary function of an expansion joint is to provide for the expansion of the pavement and thus to maintain compressive stresses within safe limits. When installed between the pavement and a fixed structure, such as a bridge, an expansion joint serves also to prevent the pavement from exerting damaging pressures against the structure.

Expansion joints also function as contraction joints and, furthermore, permit vertical angular movement of the slab ends in the manner of hinged joints. They are usually constructed with an opening of ¾ in. or 1 in., in which is installed a filler of some suitable resilient material such as premolded bituminous fiber board, wood, cork, or other material intended to provide expansion space and to prevent the infiltration of water and foreign substances. Actually, none of the materials so far used as joint filler has been able to exclude either water or foreign bodies completely. One of the most important functions of fillers, therefore, is to counteract the damaging effects of infiltrated solid materials by acting as a compressible gasket to prevent excessive localized restraint to joint closure. A major deficiency of all joint filler materials thus far employed is that they fail to recover in thickness after prolonged periods of compression and, therefore, in the course of time, fail to act as compressible gaskets. Desirable characteristics of filler material are relatively high resistance to compression, freedom from a tendency to extrude from the joints when compressed, and complete recovery in thickness after compression. The joint opening is usually sealed with a bituminous material, mixtures of bituminous material and rubber, or some mechanical sealing element. Slip dowels or proprietary load-transfer devices are generally included.

Contraction joints

Contraction joints are installed in the pavement to maintain tensile stresses within safe limits. Since contraction joints must be free to open, the continuity of the longitudinal reinforcement, if present, is interrupted at the joints. Dependence must be placed either on aggregate interlock or on load-transfer devices to maintain slab alignment.

Basically, there are two types of contraction joints, namely, the "plate-type" and the "groove-type," the latter being most commonly employed.

Plate-type—Fig. 1 shows the typical features of a "plate-type" contraction joint. This joint is formed by erecting a separator or parting strip on the subgrade. The separator, which usually consists of a metal plate, or a thin sheet of some other rigid, non-compressible material, serves to interrupt the continuity of the pavement. The groove formed

in the concrete immediately above the separator serves as a reservoir for sealing material. A load-transfer device of some kind is generally used in conjunction with this type of joint, since interlocking of the aggregate is prevented by the separator.

Groove-type—In general, this type of joint is created by reducing the effective area of the cross section of the pavement by approximately one-third, thus causing the formation of a transverse crack at the joint. This is accomplished by either installing a thin premolded bituminous strip vertically to the desired depth or by forming a groove in the surface, as indicated in Fig. 1. In addition to the surface groove, a parting strip of wood, metal, or premolded material is sometimes provided at the bottom of the slab, directly under the groove, to insure the formation of a vertical crack and to further encourage early cracking of the concrete at the joint.

Most contraction joints, especially those of the groove type, are sealed with bituminous material or, more recently, by a rubber-asphalt compound, to exclude water and solid materials. It is not general practice to seal contraction joints employing premolded bituminous strips to form the groove.

Vertical slab alignment is usually maintained by aggregate interlock or the use of load-transfer devices.

Hinge joints

The term "hinge joint" applies to any joint which permits hinge action but no appreciable separation of the adjacent slabs. Hinge joints are usually installed in conjunction with reinforced pavements and are intended primarily to control warping stresses. In contrast to contraction or expansion joints, appreciable changes in joint width are prevented by steel reinforcement or bonded steel dowels passing through the joint. In effect, a joint of this type acts simply as a hinge, that is, it merely permits a certain amount of vertical angular displacement of the abutting slabs (Fig. 1).

Hinge joints generally are constructed in the same manner as grooved contraction joints, except that in reinforced pavements the longitudinal reinforcement is not interrupted but is continuous through the joint, thereby allowing hinge action without appreciable slab separation. In the absence of reinforcement, bonded dowels are used to hold the slabs together. Load transfer is usually accomplished by aggregate interlock. For butt-type joints, other means may be employed. The joints may or may not require sealing, depending upon their design.

Construction joints

This term applies to the joint necessitated by any prolonged stoppage of the concreting operations, such as at the end of the day's work,

equipment breakdown, delays in delivery of materials, etc. It may be any one of the three types of joint—expansion, butt-type contraction, or hinge—depending upon the features of the pavement design and prevailing practice.

LONGITUDINAL JOINTS

Experience has shown that the full-width pavements constructed in the past have been highly susceptible to longitudinal cracking, the location of the cracking being essentially along the center line. As a preventive measure, it is current practice to divide the pavement into lanes less than 15 ft wide by longitudinal joints.

These joints appear at various longitudinal positions in the pavement, depending upon the width of construction, and serve to control tensile stresses. Longitudinal joints generally fall in the category of hinge joints and may be constructed in one of several ways (Fig. 1). This type of joint may be an ordinary butt joint resulting from lane-at-a-time construction, or one created under full-width construction by embedding a metal or fiber plate in the concrete along the proposed line of the joint and so shaped as to create an interlocking of the slab edges, or it may be created by weakening the pavement with a deeply formed groove, or with an embedded strip of prefabricated bituminous material. The adjacent slabs are usually prevented from separating and faulting by means of bonded steel tie bars spaced at proper intervals. The joint may or may not be sealed, depending upon the manner of construction.

FACTORS IN JOINT DESIGN

The design of joints necessitates the consideration of certain structural features which enable the joint to perform its intended function, including:

1. Allowance for movement of slabs during expansion, contraction and warping, which in turn is dependent to a certain extent upon (a) the length of slab or pavement section, (b) changes in original slab length due to autogenous volume changes and moisture conditions, (c) seasonal and daily temperature fluctuations with respect to construction temperatures, (d) subbase characteristics and (e) thermal coefficient of expansion of the concrete.

2. The insurance of mutual vertical alignment of adjacent slabs to preserve desirable riding qualities.

3. Provision for load transfer, where necessary—including aggregate interlock and mechanical devices with their supporting systems.

4. Maintenance of adequate seal against infiltration of water and foreign matter.

Joint width changes

It has been conclusively demonstrated by experimental field studies[1] that joint width changes can be controlled by proper spacing of trans-

verse joints. The amplitude of joint width changes can be narrowed to any predetermined range for a given climate by sufficiently increasing the number of transverse joints to form proportionately shorter slabs. Some compromise usually is necessary between the two alternatives of least possible width change and smallest possible number of transverse joints, both of which are desirable features.

It is generally considered that the spacing and design of expansion joints should be dependent on the allowable compressive stress in the concrete. The average compressive strength of cores from present-day concrete pavement at ages beyond 28 days generally ranges from 4000 to 6000 psi. Such strengths are well in excess of the theoretical compressive stresses resulting from thermal volume changes which the concrete will be called upon to withstand under ordinary circumstances. Furthermore, compressive stresses are reduced at early ages because of the initial shrinkage of the concrete which takes place during the early hardening period, and may subsequently be reduced by plastic deformation of the concrete under continued compression.

Just how much compressive stress relief is provided by the above phenomena is not definitely known, but from stress calculations and the performance of experimental field sections constructed without expansion joints it seems safe to say that, in most cases, the compressive stress will not reach the values on which the pavement design is based, at least for some years. This does not mean, however, that the compressive stress at any given time cannot equal or exceed the strength of the concrete in certain localized areas, such as at joints and cracks.

Studies of blowups generally reveal that there had been a progressive deterioration of the quality of the concrete itself in the vicinity of joints prior to failure. Such deterioration is a contributing factor in the successive partial failures under excessive localized compressive stresses which eventually result in complete crushing or blowup of the joint. Infiltration of foreign materials plays a very important part in this process.

Many states have now omitted expansion joints except, in some instances, during cold weather construction, or to relieve horizontal pressures at bridge structures or other critical locations. When expansion joints are specified in these states, spacings of 400 to 600 feet are commonly used. The thought back of this practice is to relieve compressive stresses but not to eliminate them entirely since it is believed that a certain amount of compressive stress is desirable.

The amount of contraction joint opening in a given climate is a function of (a) their spacing, (b) the spacing of such expansion joints as may be included in the design, (c) temperature prevailing during con-

struction and (d) the materials involved in the construction of the pavement. The maximum opening of contraction joints will, of course, be profoundly influenced by the extremes of temperature and moisture range characteristic of the prevailing climate.

There is now evidence[1] that when contraction joints have once opened they seldom return to their original width even when the pavement is under compression. This increase in residual joint opening with time is more pronounced as the spacing of the contraction joints is increased, and the spacing of expansion joints decreased. Measurements of contraction joint openings under various joint spacing conditions indicate that ultimate openings of as much as 0.30 in. may be expected for 100-ft spacings when no expansion joints are used, and 0.50 in. for 60-ft spacings with 1-in. expansion joints containing premolded bituminous filler spaced at 120-ft intervals.

Inasmuch as many engineers are reluctant to install contraction joints at the close intervals required to insure aggregate interlock, the current practice in some locations is to install the joints at intervals that range from 30 to 100 ft and to compensate for the loss in interlock by the installation of dowels or load-transfer devices.

Slab alignment

In addition to providing for expansion and contraction, as the case may be, it is the function of a joint, together with its load-transferring means, to preserve indefinitely the original riding qualities of the pavement. However, present design analysis for slab thickness does not take into consideration nonuniform subgrade conditions, subgrade volume changes caused by moisture and frost action, nor the pumping action and the subsequent faulting of slab ends resulting from decreasing subgrade support in the vicinity of joints. This latter phenomenon is associated with (1) continued consolidation of granular subgrade material by vibratory effect of traffic over joint edges, (2) by the diluting effect of surface water which filters into these areas through open joints or by seepage at the sides of the pavement and (3) plastic deformation of the subgrade under load action, considering that the deflection of the slab ends may be three to five times that of the interior of the slab.

Experience indicates that to attain a continuing satisfactory riding pavement it is necessary, as an insurance measure, to provide a suitable load-transfer device having the twofold function of transferring the loads across the joint and maintaining the mutual alignment of the adjoining slab surfaces. By the introduction of such a device the deflection of the slab ends may be reduced approximately 50 percent, and the deflection of the slab ends relative to each other almost 100 percent. To this end, joints must be designed to include, where necessary, satisfactory load-transfer devices together with appropriate holding assemblies.

Sealing of transverse joints

In spite of all care in the design of load-transfer units, or in the subsequent construction and installation of the joint assembly in the pavement, or in the final hand-finishing of the concrete at the joint, there remains one factor in the joint problem which has been sadly neglected and that is the development of ways and means for constructing a transverse joint such that it will completely exclude water and solid foreign materials throughout the life of the pavement.

To seal a joint adequately, means must be provided to exclude water and foreign matter from the top and bottom and also the ends. In the case of expansion joints, an attempt is now made to do this by inserting, into the joint opening, premolded filler materials sealed at the top, or wood boards. These materials are supposed to expand and contract in unison with the slab ends and thus keep the joint opening closed.

Joint fillers—Consideration of providing the best possible type of joint filler material for expansion joints needs stimulation. Since the primary purpose of the joint filler is to prevent infiltration of foreign matter when the slabs are contracted and to support the joint-sealing compound at the top, the material should have the greatest possible resilience combined with the property of minimum extrusion under cyclic pressures. The bituminized fiber joint fillers in common use today have excellent nonextruding properties but, on the other hand, they possess little resilience. Wood boards have been used successfully for years by some states. At the present time, compressed wood is being given attention. The wood is compressed in the dry state to required thickness. When installed in the joint it has the potential property of expanding the amount compressed, plus normal swelling when saturated. Other premolded bituminized materials are available for expansion joint fillers. Sponge rubber sheeting has been suggested for this purpose. The problem of providing a suitable filler material must yet be solved. It is believed that industry can supply such a material if highway administrators would strongly demand it and be willing to pay the price.

The bituminized fiber parting strip so commonly used to form contraction joints offers no positive seal to the joints when they are open during cold weather. Furthermore, because of its flexibility and instability, it is most difficult to install properly, even with careful workmanship. Consequently, spalling, scaling and subsequent disintegration of the concrete adjacent to the joints sometimes develop. The grooved type of joint properly sealed with a poured material offers many advantages.

Sealing compounds—Hot-poured asphalt-rubber compounds are widely used today for the sealing of all types of joints in concrete pavements.

As compared with the straight bituminous products they represent a marked advance in sealing materials, but the problem of adequately sealing joints is still far from solved. Experience now indicates that these materials are not doing an entirely satisfactory job even when applied under proper conditions. There is urgent need for improvement in these materials to produce better wetting and cohesive properties. Also, their application should not require such exact control, which is impractical to achieve under normal construction project conditions. Furthermore, there is need for considerable improvement in joint-sealing material of this or any other type, and the development of more satisfactory test procedures to insure proper quality. Asphaltic oil-latex joint-sealing material, although used to a limited extent, has shown remarkable durability in service. More recently, premolded neoprene rubber materials have appeared on the market.

The successful sealing of joints requires not only a suitable sealing material but also proper workmanship in applying it. Present specifications provide for such construction procedures, but they are seldom carried out conscientiously in the field. In other words, to insure positive adhesion of the joint-sealing material to the concrete, to effect a proper bond at all times, it will be necessary to provide methods to counteract deficiencies in the preparation of the joint surfaces and in applying the materials.

Joint shields—It is now the practice in some states to provide a shield at bottom and sides of the joints to prevent infiltration of foreign material. These shields are approximately 12 in. wide and are made of sheet metal or premolded bituminous fiber board. Insufficient time has elapsed to form any definite conclusions as to the merit of this practice.

LOAD-TRANSFER DEVICES AND JOINT ASSEMBLIES

As previously indicated, load-transfer devices have a dual function, namely:

1. To prevent differential vertical displacement of the adjacent slab ends—such displacement being defined as "faulting" or "stepping-off."

2. To mechanically transfer loads across the joint space to the extent that both slab ends will act jointly in supporting a load that is applied to only one of the slab ends—the result being a reduction in the magnitude of the deflections of the slab ends and a consequent reduction in the magnitude of the stresses induced in the pavement.

In recent years, due to the tremendous increases in both the weight and number of trucks, the action of traffic has become a factor of utmost importance in the design of joint devices. It should be understood, however, that the introduction of a load-transfer device of sufficient strength to cause both slab ends to deflect equally and simultaneously

under the action of traffic will not necessarily result in a pavement that has uniform load-carrying capacity throughout its entire length. It has been demonstrated in actual practice that the pavement at joints and cracks is far more susceptible to failure than elsewhere, and it is for this reason that the proper design of joints and the prevention or control of cracking warrant serious consideration.

Design of load-transfer devices

Load transfer as a factor in joint design was first recognized in 1917 when, on a concrete pavement project near Newport News, Va., steel dowels were placed across all transverse joints for the stated purpose of transmitting loads across the joints by shear. Since that time numerous patents have appeared covering all imaginable kinds of load-transfer devices. In most cases they were so expensive or impractical to construct that they have never been used.[2]

Although methods to provide structural interaction of adjoining slabs were used as early as 1917, it was not until 1928 that attention was given to the determination of spacing or to the evaluation of the effectiveness of such devices. In that year, H. M. Westergaard[3] presented a rational theory for joint design under the title, "Spacing of Dowels." Several investigators[2,4,5,6,7,8,9] have since presented theories and methods of tests for the evaluation of load-transfer devices, together with results of field and laboratory studies.

The results of these investigations, particularly in regard to the functioning of load-transfer devices, indicate that there exists, in some cases, a close agreement between field tests and values obtained by theoretical calculations based on the Westergaard theory of slab deflections. A thorough study of these investigations is encouraging and gives evidence that a rational method of joint design can be established whereby a definite relationship between the load-transfer device and the characteristics of the slab and subgrade may be utilized.

An examination of the load-transfer devices which have been used or are in use today reveals certain definite principles in their design. These principles may be classified as follows:

1. The development of shear resistance with hinge action.
2. The development of shear and bending moment. (The most common unit in this class is the conventional dowel bar.)

Further, a review of load transfer practices for highway pavements[10] brings out other interesting facts, namely:

1. The conventional round dowel is used to a greater extent than any other type of load-transfer device. The diameter ranges from ¾ to 1¼ in., and the length from 12 to 33 in.
2. The spacing of the dowel bars ranges from 12 to 16½ in. center to center. Proprietary devices range in spacing from 12 to 20 in.

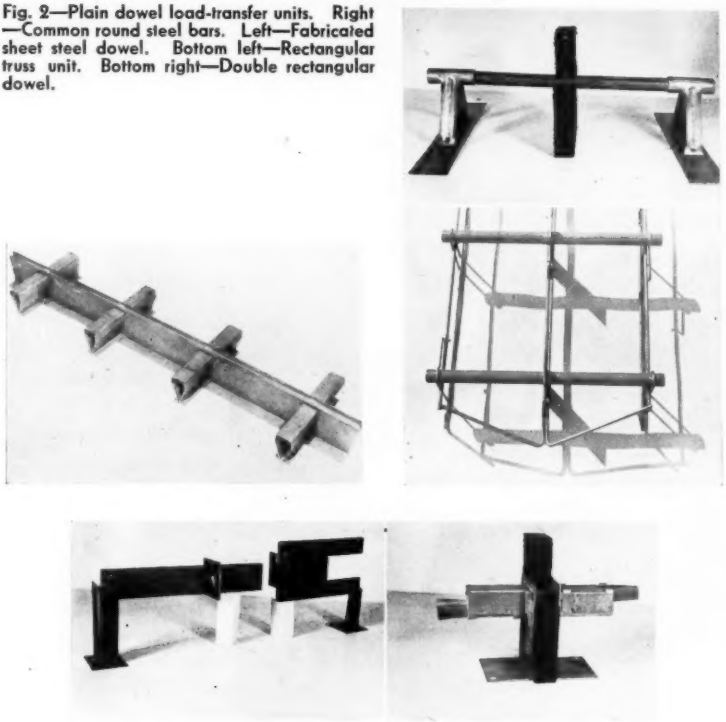


Fig. 2—Plain dowel load-transfer units. Right—Common round steel bars. Left—Fabricated sheet steel dowel. Bottom left—Rectangular truss unit. Bottom right—Double rectangular dowel.

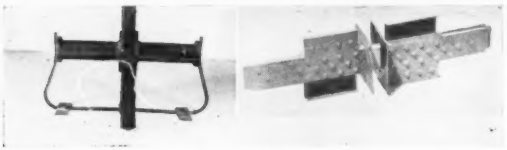
3. The installation of some form of load-transfer device is standard practice in most states, but there are some states that do not install load-transfer devices of any kind.
4. Several states permit the use of approved proprietary load-transfer devices.
5. Some states use load-transfer devices only of their own design.

This information is presented to show the variation which exists in present-day design requirements for load-transfer devices, and to further show the need of a method for evaluating devices on the basis of their ability to perform their intended function.

Methods of effecting load transfer

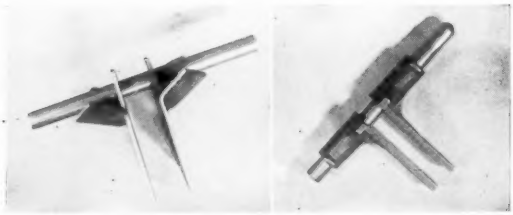
Various types of load-transfer devices or methods of effecting load transfer will be discussed and illustrated in accordance with the basic principles involved in their construction and use.

Dowels—Plain round steel bars have been most extensively employed to dowel the ends of slabs together. Dowels are not, however, limited



Left—Dowel socket unit
Right—"A" dowel

Left—H-Bar
Right—J-Bar (new)



Left—Wing bearing unit
Right—J-Bar (old)

Fig. 3—Dowel type load-transfer units with metal sleeves

to round sections, inasmuch as various structural sections such as rectangular bars, channels, I-beams and tubing have been employed. In this type, the dowel and concrete are in direct contact. Several well-known types are shown in Fig. 2.

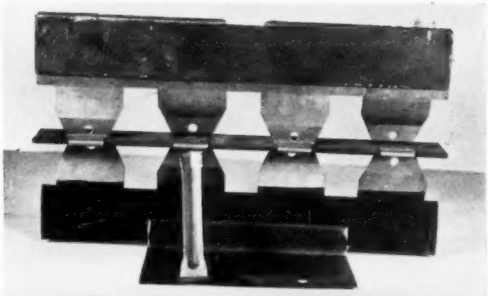
Dowels with metal sleeves—Load-transfer units in this class have metal sleeves surrounding the dowel bar. The sleeves become embedded and anchored in the concrete. The function of the sleeves is to distribute the stresses into the concrete surrounding the load-transfer unit, thus preventing the highly concentrated bearing stresses that the dowels would otherwise exert on the concrete. Several devices employing this principle are shown in Fig. 3.

Plate dowels—Load transfer by plate dowels is effected by a steel plate which extends the entire length of the joint. The plate often is encased in a metal shield to promote freedom of movement. Examples of this type are shown in Fig. 4.

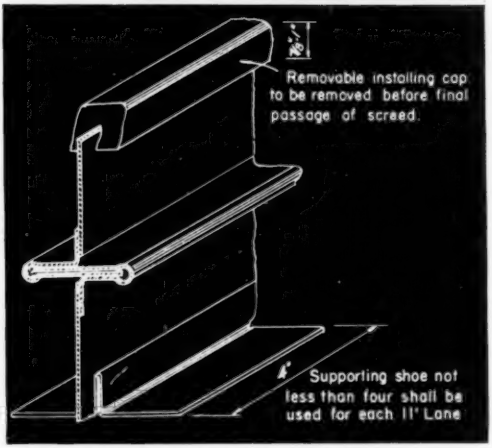
Hinge action units—These units are designed to effect load transfer through direct shear, with no bending moment. A typical example is shown in Fig. 5.

Lug units—These are generally made of malleable iron castings shaped in such a manner as to become firmly embedded in the concrete.

Fig. 4—Plate dowels



Key lode


Removable installing cap to be removed before final passage of screed.
Supporting shoe not less than four shall be used for each 11' Lane

Continuous plate

Short lugs are appended which serve to transfer the load across the joint through direct shear action. A common type of lug unit designed for use in expansion or contraction joints is shown in Fig. 6.

Ledge construction—This refers to methods of effecting load transfer without metal devices, by special construction of the slab ends at the joint. Such methods usually consist of some type of ledge construction which transmits load through shear action, but which permits free horizontal movement of the abutting slabs. Joints of this type are illustrated in Fig. 7.

Sill joints—The typical features of a sill joint are shown in Fig. 8. Actually, the concrete sill cannot properly be termed a load-transfer device, or in fact a device of any kind, since it is simply a means of providing a greater degree of support for the slab ends than would be provided by the subgrade alone. The effect of the sill is to distribute

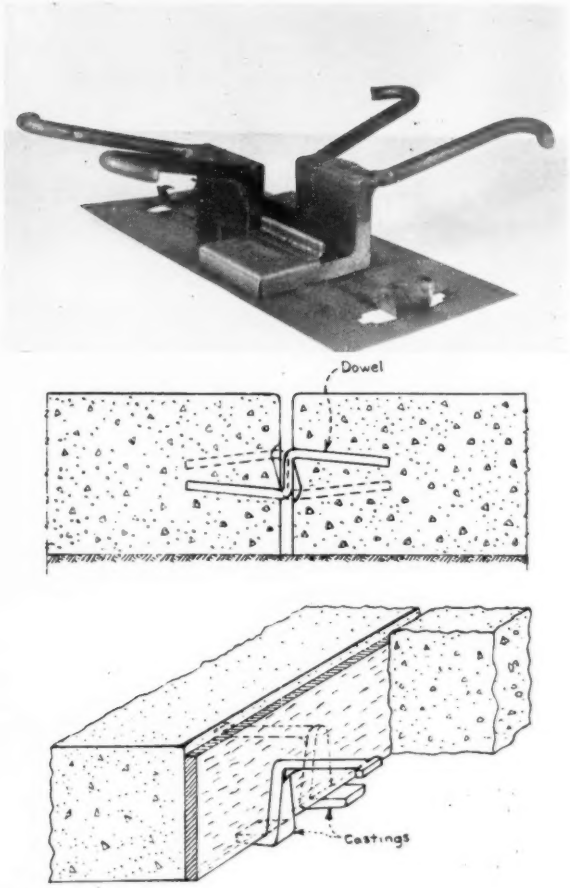


Fig. 5—Hinge action units. Top—Translode angle. Center—Z-Bar. Bottom—Spade unit

applied loads over a greater area of subgrade and thus reduce the unit-bearing pressures on the subgrade. Consequently, to be effective, the sill must necessarily have considerable surface area. In Fig. 8 the upper surface of the sill is shown flush with the subgrade surface. However, in an effort to counteract the possible infiltration of earthy materials between the sills and the pavement, the tops of the sills have in some instances been constructed from 1 to 2 in. above the subgrade surface,

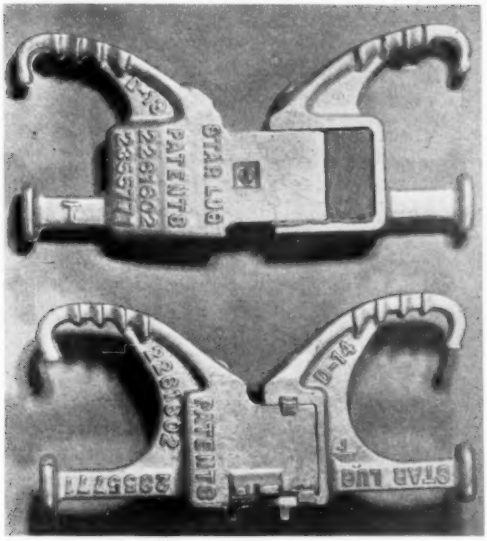


Fig. 6—Lug units

in which case they are partially inset in the pavement. In some locations, as a supplemental feature, sills have been employed in conjunction with joints having load-transfer devices.

Requirements for load-transfer devices

A satisfactory load-transfer unit must be designed and constructed in accordance with certain principles. It is believed that these principles are:

1. The units should be economically justifiable.
2. They should be simple in design so that they may be practical to install, and permit positive encasement by the concrete.
3. They should be capable of distributing the load stresses throughout the adjacent concrete in such a manner that these stresses will not exceed the allowable design value. In this respect it is especially important that high localized stresses in the concrete at the joint face be prevented.
4. They must offer no material restraint at any time to the opening of the joints.
5. They must retain their mechanical stability under wheel load weights and frequencies comparable to those for which the pavement itself has been designed.
6. They must be constructed in such a manner as to meet specified performance requirements relative to load transfer capacity.
7. They should be resistant to corrosion.

The load-carrying capacity of a load-transfer assembly is dependent upon several factors of which the more important are size, length and

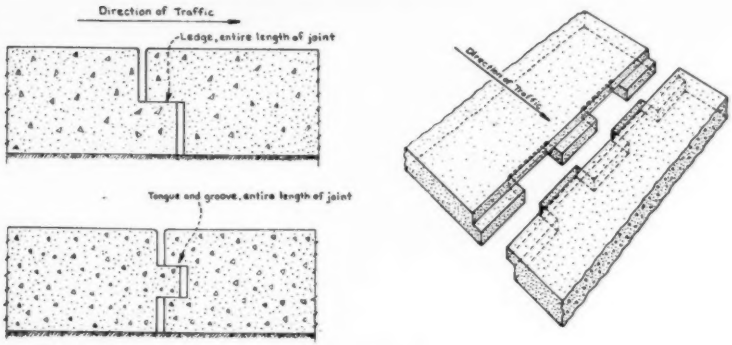


Fig. 7—Ledge construction

spacing of the dowels, or the mechanical efficiency of nondowel types, width of joint opening, thickness of the slab, respective strengths of metal and concrete, stiffness of the pavement-subbase system and proper alignment of the load-transfer units. During recent years studies have been made, and are presently being made, with the view of developing a satisfactory method for evaluating load-transfer devices in terms of load transfer capacity, mechanical stability and relative effect upon stress distribution in the adjacent concrete.

In the evaluation of any type of mechanical load-transfer device it will be necessary first to establish recognized standard requirements, the most important being a definite rigidity value (joint modulus or modulus of rigidity) under specific controllable conditions. Included in the standard requirements should be the ability of the unit to reduce joint edge stresses to a required amount, also the ability of the unit to prevent faulting under the action of present and future heavy commercial traffic. The unit should also be examined for its influence on longitudinal movement. Finally, standard test procedures should be

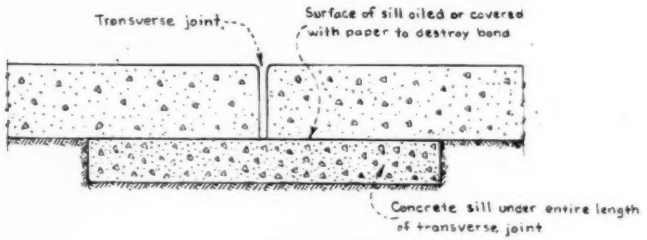

Transverse joint
Surface of sill oiled or covered with paper to destroy bond
Concrete sill under entire length of transverse joint

Fig. 8—Sill joint

developed to determine whether the units comply with the established requirements. When these characteristics of any load-transfer unit have been determined it will then be possible to specify its proper spacing in a joint assembly.

Joint assemblies

In the design of joints, the various methods used to hold the load-transfer units in proper position must be given careful consideration. The general use of dowel bars has brought out many different assemblies to hold them in place during concreting. However, the so-called dowel-bar basket is perhaps the type most commonly used. Certain proprietary makes of load-transfer units depend upon the joint filler, or a special joint plate, to hold them in position.

It is believed that a joint assembly should possess the following essential features:

1. It should be so constructed that the load-transfer units will be held firmly in place, and in their proper position.
2. It should have sufficient rigidity to maintain the load-transfer units in correct alignment, within reasonable limits, throughout the entire concreting operations.
3. The design should be such that it can be readily assembled by the contractor on the job, and installed with a minimum of difficulty.
4. It should be sufficiently rigid to permit handling as a unit.
5. It should not offer any material restraint to the horizontal movement of the slabs.

At present, the ability of a joint assembly to meet the requirements previously outlined is judged primarily on visual observation, the so-called "jump test," and a series of field-check measurements on dowel bar alignment made after the assembly has been surrounded by the fresh concrete. Thus far, no recognized test method has been developed whereby the various types of joint assemblies may be examined on a comparative basis under controlled conditions simulating actual practice.

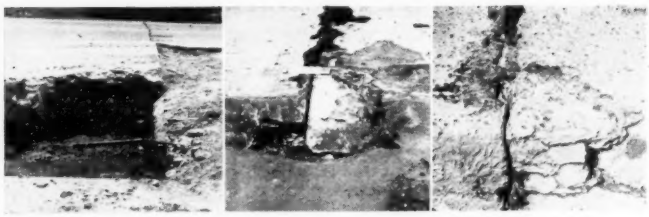
Mechanical installation of dowel units

Machines have been constructed to install dowels without the use of baskets or other assemblies. Machines of this type have been used extensively in airport work and to a limited extent in highway construction. The suitability of this method of dowel installation requires further study.

VARIOUS FORMS OF JOINT FAILURE

Past experience has shown that the pavement adjacent to joints is susceptible to various forms of failure. The more common forms are usually faulting, pumping, crushing of the concrete, blow-ups, spalling, scaling, longitudinal cracking and transverse cracking adjacent to joint edges (Fig. 9).

Fig. 9—Various forms of joint failure



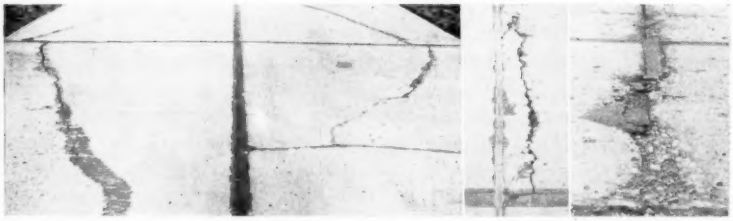
Blowup

Typical end spalling

Spalling at center joint

Scaling along joint

Spalling due to tipped joint



Longitudinal cracking—Infiltration at edges

Partially locked joint

Crushing of concrete

Faulting

Permanent depression of the approach slab at joints is by far the most serious and widespread form of joint failure and its prevention in future construction is a problem of major importance to the highway engineer. It is generally attributed to a lack of subgrade support and inadequate load transfer requirements in association with heavy commercial traffic—in short, to inadequate pavement design in terms of present-day load conditions.

Pumping

A certain amount of pumping action probably takes place at all joints, but the type of pumping failure referred to is associated almost

exclusively with pavements constructed directly on fine-grained impervious subgrade soils, such as clays, silts, loams and combinations thereof. With water present, the vertical movement of the slab ends under heavy loads causes a progressive pumping-out of the subgrade soil which ultimately results in slab failure, as manifested by faulting or permanent depression of the pavement at the joints and cracking of the pavement.

Crushing of the concrete

Crushing of the concrete in the vicinity of joints under high compressive forces may be the result of several factors acting either individually or concurrently. It may be due to inferior concrete caused by improper mix design, poor workmanship, or the use of inferior materials. Crushing may start through the development of localized pressure at the joint faces due to the presence of inert material such as small stones, pieces of concrete left from construction operations or compacted soil materials. Incipient cracks caused by the above conditions, and in some cases augmented by freezing and thawing action, ultimately lead to major failure of the concrete.

Blow-ups

The two principal causative factors in blow-ups are believed to be infiltration of foreign materials into joints and cracks, and abnormal permanent growth of the concrete in the pavement. The presence of foreign bodies not only reduces available expansion space but also causes highly localized compressive stresses which eventually result in spalling, cracking and general progressive weakening of the adjacent concrete. Abnormal permanent growth of the concrete also will eventually exhaust the available expansion space and set up permanent compressive stress at the joint faces.

Either or both of these factors may operate to cause blow-ups during periods when the stresses already existing are greatly augmented by certain weather conditions. The effects of both factors are aggravated by antecedent defects, such as inferior concrete, nonvertical joint faces or spalling from other causes.

Spalling

Localized spalling of the concrete parallel and adjacent to the edges of joints is generally attributed to several factors associated with the construction of the joint and to the relative warping action of the abutting slabs. In the first case the spalling can be attributed to poor workmanship. Considerable spalling can also be attributed to the tipping and misalingment of the groove-forming strip by the finishing machines. Also, failure to remove thoroughly all obstructions from joint grooves prior to sealing them is the source of much spalling. The

subsequent infiltration of foreign material into the joint during periods of contraction is another cause of considerable spalling.

Scaling

In time, localized scaling along joint edges may develop. Overfinishing of the concrete at the joint in an attempt by the workmen to produce a nice appearing job is an important cause of this condition.

Longitudinal cracking

Longitudinal cracking starting at transverse joints usually is caused by the localized infiltration of solid materials from the shoulders into the joint spaces, thus setting up unequal compressive stresses during an expansion cycle.

Transverse cracking near joint

Parallel and irregular transverse cracking of the pavement adjacent to joints is, in many cases, caused by excessive restraint to the opening of the joints. Furthermore, any such restraint tends to cause failure of the reinforcing steel at the points where it passes through transverse cracks.

The improper and careless placing of joint assemblies is known to cause diagonal cracking and spalling in the immediate vicinity of the joints.

Certain types of load-transfer devices also are known to have caused transverse cracking and spalling in these areas.

Locked joints

Conditions resulting in restraint to joint opening are: mechanical imperfections in dowels, such as surface roughness, variable cross section, end burrs and crookedness; misalignment of the units; faulty lubrication and corrosion.

The extent to which mechanical imperfections and insufficient lubrication of load-transfer devices has been responsible for locked joints is not known, but laboratory studies to date definitely indicate that these two conditions can materially affect the movement at joints and that they are factors which should be given serious consideration in future joint design.

Little appears to be known with regard to the corrosion of load-transfer devices and the effect that it might have on pavement performance. It is known, however, that corrosion can and has resulted in a serious reduction of cross-sectional area of dowels at the point where they pass through the joint space. Moreover, it has been known to result in the seizure of large dowels, with resultant opening of cracks and failure of the reinforcing steel at the cracks.

It is common knowledge that ordinary steel, if unprotected and exposed to the elements, usually will corrode rapidly. That this fact has not been generally appreciated in connection with load-transfer devices, which necessarily are obliged to function under conditions highly conducive to corrosion, is rather difficult to explain.

A few typical examples of common joint failures which may be traced to one or more of the factors enumerated above are presented in Fig. 9.

In general, it can be stated that most joint failures can be directly or indirectly traced to one or more of the following conditions: (1) lack of subgrade support, (2) inadequate load transfer, (3) unstable joint assemblies, (4) inferior concrete, (5) unsatisfactory sealing methods, (6) faulty lubrication, (7) corrosion and (8) poor workmanship. All of these can be corrected to a considerable degree by proper action on the part of those responsible.

TRENDS IN JOINT DESIGN

Recent condition surveys of concrete pavements constructed prior to and during the late war have revealed that the then normally accepted design for joints has proved inadequate for present and future traffic conditions. Consequently, since the war, there has been a decided change in the attitude of some highway administrators toward providing better jointing with less regard to cost. It should be of interest to mention the major trends in joint design and construction practices which are now taking place.

It has been sufficiently demonstrated, and now generally recognized, that under heavy truck traffic some type of mechanical load-transfer device is essential to the preservation of the structural integrity of the pavement, and to the maintenance of desirable riding qualities. To accomplish this there is a definite trend toward the use of larger diameter dowels at closer spacing. The ¾-in. dowel is rapidly being replaced with ⅞- to 1¼-in. dowels spaced at 12- to 15-in. intervals. Dowel length still varies considerably. Research indicates that 15 to 18 in. is satisfactory. Experience indicates that on heavy trucking routes, the larger dowels are necessary.

Considerable research is under way to determine the load-deflection characteristics of various types of load-transfer units under static and dynamic loads. This work should provide basic information on which to prepare specification requirements for the performance and spacing of load-transfer units in a joint.

Advances are being made in the fabrication of steel dowel-bar baskets and other devices utilized to hold load-transfer units in proper position.

One state is now using shields at the bottom and ends of joints, to

prevent the infiltration of foreign matter. Other states are experimenting with the idea, but it is too early to evaluate the merit of this design.

The molded groove for creating the plane of weakness at contraction joints is fast replacing the premolded fiber board parting strip. These strips have fallen into disfavor because of the surface failures associated with their use.

The hot-poured rubber-asphalt sealing compounds are fast replacing the ordinary asphalt or tar products used extensively in the past for joint and crack sealing. When properly applied, they are far superior to the older products.

The use of uniform, well-consolidated granular base courses under pavements is now standard practice in many states. This practice, which is on the increase, should help materially in preventing several forms of serious joint failure common to older pavements.

The use of air entrainment, which will very likely provide better concrete performance at joints, is on the increase.

There is a definite trend toward the omission of expansion joints, but it is still too early to predict what effect this practice will have on the ultimate behavior of joints and concrete pavements in general.

SUMMARY

In summary, the above review of the joint problem discloses certain factors in present-day concrete pavement practices which ultimately affect the performance of joints. It is evident that a concerted effort should be made by those concerned to encourage better joint practices by giving express consideration to the following:

1. Raise the general standard of workmanship through more critical inspection, together with the proper backing of those charged with inspection.
2. Construct uniformly consolidated subbases of suitable materials under all pavements.
3. Encourage the development of specifications for the design, evaluation and placement of load-transfer devices and joint assemblies.
4. Use these specifications to provide, within the limits of economy, the best type of load transfer possible.

ACKNOWLEDGMENT

The authors wish to express their sincere gratitude to L. W. Teller for his review and valuable criticism of the work; and to the members of the committee—Bengt F. Friberg, Nathan M. Newmark and C. A. Willson, for their helpful assistance.

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Structural Design of Joints for Airport Pavements

By THOMAS B. PRINGLE

The preceeding information on design and construction of joints in concrete pavement surfaces is supplemented by a discussion of jointing practices used by the Corps of Engineers, U. S. Army, in constructing pavements for heavy wheel loads. The first part gives Corps of Engineers' practice as published in the *Engineering Manual*, Part XII, Chapter 3, dated July 1946; the second part gives some of the background and reasons for the Corps' practice.

JOINT PRACTICE FOR HEAVY DUTY PAVEMENTS

The Corps of Engineers' practice requires three general types of joints; namely, weakened plane or dummy type, construction and expansion. Some variations in the designs and uses of these joints are provided.

Dummy or weakened plane joints

Dummy or weakened plane joints are installed 12½ to 25 ft apart. The dummy joint is constructed to extend one-fourth the depth of the slab. Deformed steel tie bars, ⅝ in. in diameter, 2 ft 6 in. long and spaced 30 in. on centers, are used in joints adjacent to free edges or ends of paved areas.

Longitudinal dummy joints are placed along center lines of strips of pavement (paving lanes) which have a width of 16 ft or more if designed for 60,000-lb single wheel loads or less. For pavements constructed in narrower strips or for greater wheel loads, no longitudinal dummy joint is required.

Transverse dummy joints are placed across each paving lane perpendicular to the center line, at intervals of not less than 12½ ft and not more than 25 ft, with the spacing of joints made uniform throughout any major paved area. Each joint must be straight and continuous from edge to edge of paving lane. Staggering of joints in adjacent paving lanes is not permitted. Under ordinary conditions of subgrade and climate, the following spacings, corresponding to the various listed types of concrete aggregate, are used. However, where practicable, advantage is taken of available information regarding local conditions and probable sources of aggregate.

Aggregate	Transverse joint spacing, ft
Granite	20—25
Limestone and dolomite	20—25
Flinty material and quartzite	15—20
Slag	12.5—15

Exception is made in the case of thicker slabs (designed for wheel loads greater than 60,000 lb). These are placed without longitudinal dummy groove joints. The spacing of transverse joints in the thicker pavements is equal to the width of paving lane and will usually be 20 to 25 ft. Dowels are not required across dummy joints. Tie bars are required only across joints next to free ends or edges of paved areas.

Construction joints

Longitudinal construction joints are generally 20 to 25 ft apart, depending usually upon the size of paving equipment. Doweled longitudinal joints usually are preferred and all transverse construction joints are doweled. The extreme edges of all paved areas, because of the possibility of being extended, are thickened or formed so as to provide regular construction joint installation.

Expansion joints at structures and intersections

Expansion joints consisting of approved preformed bituminous filler or wood are installed to surround or to separate new pavement from all structures and features which project through, into, or against it. The thickness of filler and manner of installation depend upon the particular case. Usually ¾-in. thickness is sufficient. Expansion joints are employed if strips of pavement or extensive paved areas intersect or are connected in such manner that expansion of the concrete would be expected to cause serious distortion, buckling, or displacement.

Expansion joints within pavement

Expansion joints are placed at regular intervals of 400 ft throughout large paved areas such as runways, aprons or taxiways for the thinner pavements (those with slab thickness less than 10 in.). Regularly spaced expansion joints are omitted in the thicker pavements (those 10 in. or more in thickness) unless the concrete is composed of aggregate particles which have high coefficients of thermal expansion. If the major portion of the concrete aggregate is siliceous material or some other composition which has high thermal expansion (0.0000055 in. per °F or greater), the pavement, regardless of slab thickness, is provided with expansion joints spaced not more than 400 ft apart.

Use of dowels

Dowels are required at all transverse expansion and construction joints and are usually preferred for longitudinal construction joints. Dowel size is determined from pavement thickness. Table 1 gives Corps of Engineers' practice.

Edge thickening at free joints

Thickened edges are used with longitudinal expansion joints and pavement edges which are not formed in a manner to provide regular

TABLE 1—REQUIREMENTS FOR USE OF DOWELS*

Pavement thickness, in.	Dowel diameter, in.	Minimum dowel length, in.	Dowel spacings, in.	Type of dowel
Less than 8	¾	16	12	¾ in. ϕ bar
8 to 11	1	16	12	1 in. ϕ bar
12 to 15	1¼ to 1.3	20	15	1¼ in. ϕ bar or 1 in. extra strong pipe
16 to 20	1¾ to 1.9	20	18	1¾ in. ϕ bar or 1½ in. extra strong pipe
Over 20	2¼ to 2.3	24	18	2¼ in. ϕ bar or 2 in. extra strong pipe

*Dowels are not used in longitudinal expansion joints.

construction joint installation. The thickening is approximately one-third of the designed thickness. The thickening extends from the center of the slab to the free edge in instances where the longitudinal dummy is used; and for other slabs, the thickening is provided across the paving lane.

BASIS FOR JOINT PRACTICE

The Corps of Engineers' practice is based on observations of existing pavements and investigations conducted by the Corps of Engineers' Rigid Pavement Laboratory. This discussion is based on these and other observations.

The loads imposed on pavements from the increasingly heavy military aircraft involve some problems in regard to the functioning of joints between slabs of concrete pavements which are somewhat different from the requirements of highway pavements. However, the structural designs presently used by the Corps of Engineers in airfield construction do not depart materially from present highway practice except dowels are usually preferred for transverse expansion and all construction joints.

The preferred use of dowels for transverse expansion joints is based on results from accelerated traffic tests in which some ten expansion joints were constructed alternately with and without dowels. They were subjected to wheel loads of 37,000 lb and 60,000 lb at the same time throughout variable seasons of the year. Concrete at the joints without dowels consistently started to crack before the concrete at the joints having dowels. There still is considerable question as to whether dowels, in types of joints other than expansion joints, show the same advantage under aircraft traffic but there seems to be an indication that

a more balanced over-all pavement design might result by the general use of dowels at joints. This is especially true if pavements are overloaded.

A more definite conclusion based on traffic tests, seems justified for the use of doweled construction joints. Tests were made on pavements ranging from 15 to 24 in. thick utilizing 16 different types of joints ranging from dowelless dummy joints and common doweled joints to elaborate load transfer structures. Patented joints were not included. The test pavements were tested under identical traffic coverage of a 150,000-lb wheel load during the same weather conditions. After 2000 complete coverages no appreciable difference could be detected in the distress at the various types of joints except at the keyed joints. The concrete at these joints repeatedly spalled and cracked off the lip of the keyway. This appears to be caused by the wedging of the fit between the key and keyway when the slab curls from temperature or moisture variation and the reduced cross sectional area at the location of resultant stresses. This performance is leading toward the abandonment of keyed construction joints. It may be possible to improve the efficiency of keyed joints through a change in design. Some work has been done along this line by the Rigid Pavement Laboratory but a solution is not in sight at this time.

There are several further differences between runway and highway requirements which have been investigated or are under investigation. While these have not reached the stage of standardization for Corps of Engineers' practice they are given consideration in individual designs.

Tilting of slabs

The ratio between the live load and dead load of a concrete pavement slab is greater for runways than for highways. Aircraft wheel loads have been increased but the horizontal dimensions between contraction joints, depending mostly upon the weather cannot be increased proportionately. The thickness and therefore the weight of the slab required for strength is increased, at the most, only in proportion to the square root of the increased wheel load. This situation introduces the problem of preventing the tilting of the slabs as a whole when the relatively heavy wheel load is at an edge or joint. The load transfer medium of the joint at the edge of the slab opposite from the loaded edge may be required to function in reverse, helping to hold the slab down instead of helping to hold it up. Observations and tests have shown that instead of helping to hold it up, the subgrade sometimes exerts appreciable tensile force toward prevention of the upward movement of the slab and is of substantial help in preventing tilting. In view of the problem of tilting, the general practice for heavy loads in recent construction has been to space the joints about 25 ft apart although design

computations indicate that relief of temperature stresses could be secured by closer spacing.

Consideration of blow-ups

It has been recognized that a blow-up can be more disastrous on a runway than on a highway. Runway traffic cannot be detoured readily and is at higher speeds. Moreover, a blow-up might escape immediate detection. On the other hand, runways are near level and the pavements are thicker so that the causes of buckling are greatly reduced. In view of this, the thicker runway pavements are constructed without joints. An example is the pavement for the B-36 single wheel landing gear at Wright-Patterson Air Force Base (multiple wheels are now used) which is 21 in. thick, approximately two miles long, and was constructed without expansion joints.

Effect of curved temperature gradient

Measurements of temperatures through the depth of airfield slabs 14 and 24 in. thick show that the gradient of temperature from top to bottom is not uniform. Where the gradient of temperature from top to bottom is approximately uniform, as has been proved for relatively thin slabs, say 8 in. thick, the temperature stresses can be relieved by joints. When the temperature of the concrete as a mass decreases, joints permit the concrete to decrease in volume and thereby relieve stress. When the temperature differs from top to bottom of a slab in a straight line gradient, and the slab is free to change its shape by curling, the stresses are again relieved. But when the temperature gradient from top to bottom is curved, there is no form, even with a system of joints, into which the volume of concrete can alter itself to relieve these inherent stresses. Computations based on the measured temperatures throughout the depth of slab, and assuming a modulus of elasticity of the concrete to be 3,000,000 psi, indicate that these inherent temperature stresses alone may reach 100 psi in tension and perhaps considerably more. The computations are uncertain because it is not known what proportion of yield is elastic and what portion is plastic. However, presence of these inherent temperature stresses is considered a problem in the design of thick slabs and their joint systems.

It has been observed that interior portions of the 21 and 25 in. thick concrete slabs at Wright-Patterson Air Force Base split off on the horizontal plane along the mesh reinforcing about 3 in. below the surface. The inherent temperature stresses may have been a contributing cause.

Elasticity of joint medium

General practice in theoretical design analyses is to derive the slab curvature from the elastic properties of the concrete as represented

by the modulus of elasticity and Poisson's ratio and the elastic property of the subgrade material as represented by the subgrade modulus. Model tests and theoretical analysis made by the Corps of Engineers' Rigid Pavement Laboratory now indicate that the elastic property of the joint medium has an important influence on the slab curvature. Also, in theoretical design analysis past practice has been to assume that the ratio of the stresses between the loaded slab and the adjacent slab across the joint is proportional to the ratio of the corresponding maximum deflections of the two slabs. The Rigid Pavement Laboratory studies show that stress relief is only about half of that indicated by the ratios of deflections usually assumed. A mathematical analysis of the effect of joint elasticity, devised by the laboratory, has indicated that the elasticity of different types of joints as presently used resulted in a reduction of stress due to applied load of about 25 percent of the corresponding stress at a free edge, regardless of the wide difference in shear resistance properties of the various load transfer means. Stress computations for the design curves shown in the *Engineering Manual*, Corps of Engineers, July 1946, were based on 25 percent stress relief at the joints or a reduction of stress in the loaded slab to 75 percent of the stress in a free edge slab.

Under the sponsorship of the Corps of Engineers, a basic theoretical analysis of the effect of joint elasticity has been completed at Harvard University under the direction of H. M. Westergaard, which verifies the original findings of the Rigid Pavement Laboratory. Model tests have been performed to check the results and confirm the need in rigid pavement stress analysis to give consideration to the interdependence of elastic properties of the concrete, the subgrade soil and the load transfer medium.

The above discussion of a theoretical analysis is not intended to imply that a revised approach will account for stresses due to non-uniform support, temperature or moisture. These stresses must for the most part be included in a design factor since present knowledge is not sufficient to permit a positive theoretical approach.

Discussion of a paper by William Van Breemen and E. A. Finney:

Design and Construction of Joints in Concrete Pavements*

By H. W. RUSSELL and AUTHORS

By H. W. RUSSELL †

The authors show clearly the difficulties encountered when one attempts to include in a concrete pavement a joint which will do everything a good joint should. The problems involved are numerous and complex, and volumes have been written on the subject.

Since a patent was granted about 80 years ago for materials to be used as a pavement joint filler, we know that early designers gave consideration to the use of joints. Since that time many engineers have devoted much of their time and talents in an effort to develop a satisfactory joint, including seals, fillers and load transfer devices, but even so it seems that we do not have one that is universally satisfactory. Had the same efforts been used in trying to develop a satisfactory jointless, or near jointless pavement, we might now have an entirely different story to tell. Perhaps we have been too joint conscious.

Of the thousands of joints in Illinois pavement the writer is unable to say that a single one is entirely satisfactory. However, thousands of them are *not* satisfactory. It is well known that joints lose much of their effectiveness because of the infiltration of foreign material, solid and liquid, in both joints and cracks. The ability of an open-poured joint to perform satisfactorily may be improved of course by giving it, periodically, a thorough cleaning and repouring. This, however, would be impractical and more costly than the highway purse, with its dwindling dollar, could stand.

In a winter survey made a few years ago, with a majority of about 1800 joints examined being 1 to 6 years old, 7.5 percent were found to contain foreign material having an average depth of more than 2 in. More than 200 of the open 4-in. joints included in the survey, from 4 to 16 years old, were almost completely filled with dirt.

*ACI JOURNAL, June 1950, *Proc.* V. 46, p. 789. Disc. 46-59 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 22, No. 4, Dec. 1950, Part 2, *Proceedings* V. 46.

†Engineer of Materials, Illinois Division of Highways, Springfield, Ill.

That survey included over 500 fiber joints, almost 1000 air-chamber joints and 50 premolded bituminous joints. The pavements containing fiber joints also contained wire mesh reinforcing; those containing air-chamber joints were not reinforced. The average width of the fiber joints was practically the same as when installed and the air-chamber joints had closed from about a third to a half an inch. For the entire survey the joints had lost on an average about one third of their original width, the average expansion space decreasing progressively with the age of the pavement. The approximate rate of closure of the air-chamber expansion joints was found to be about 0.10 in. per year. At that rate the original expansion space of 0.75 in. would have been taken up after about eight years, after which trouble could be expected. The results also indicated the value of suitable reinforcement in retarding and possibly preventing the infiltration of foreign material in the cracks.

A comparison of premolded fiber and premolded bituminous fillers showed that at the end of one year the fiber fillers in one third of the joints had deteriorated sufficiently to permit the entrance of dirt and water while only 2.5 percent of the comparatively small number of premolded bituminous fillers inspected were in that condition. But the latter are far from perfect also. We have found premolded bituminous fillers disintegrated within six years, with as much as 3 in. of the depth of the joint replaced with dirt.

Except to prolong the agony of decreased joint efficiency and to increase maintenance costs, what value has a seal or filler that will not bond itself to the concrete sides of a joint, or otherwise completely close the opening long enough to justify its use? We have never seen one that would really do that, and still we have to use them. Perhaps their failure to fulfill our exacting requirements may in many cases be due in part to the methods used to obtain a good seal rather than to the material itself.

It is not alone sufficient to design a perfect joint, even if that were possible. Such a joint would lose a large percent of its efficiency, even before the concrete hardened, if it were not properly installed. And we venture to say that more joints have been installed improperly than otherwise. As a result of that alone, not considering other common defects, it is quite probable that a large portion of the joints now in use are not functioning as they should. On too many construction jobs joint efficiency has literally been murdered by poor design, negligent engineers, indifferent contractors—who think mostly in terms of production—and by poor workmanship. Under those adverse conditions we look upon a joint in a concrete pavement as being akin to a cancer, and those conditions have existed too long.

The authors stated that most spalling of the concrete adjacent to the edges of joints can be traced to poor workmanship in constructing the joints. They have told us that blowups at joints are usually associated with either spalling, inferior concrete or nonvertical joint surfaces.

Last year our maintenance engineer reported that prior to Aug. 1, 3690 blowups occurred in this state in about 12,000 miles of pavements, practically

all of them in June and July during a period of high temperature, precipitation and humidity. About two-thirds or, approximately 2500, required replacement with new concrete. In all probability a few others occurred later but were not reported.

We do not have the exact number of blowups occurring at joints but we know that a large number of them did occur at or near a joint. They occurred in practically all types of pavements containing a wide variety of joints and joint spacings. From this it would seem that joints alone are not sufficient insurance against blowups.

Even our more recent design is not free from blowups. One is known to have occurred last year in a 9-in. pavement, laid in 1946, with 78-lb wire mesh and contraction joints spaced 100 ft apart, and there may have been others.

Joints add so many problems to the design, construction and maintenance of pavements that we sometimes think they cause more headaches than they are intended to relieve. Certainly, as pointed out by the authors, joints are points of structural weakness and form potential channels for the movement of water to and from the subgrade. As usually installed they detract from the riding qualities of pavements, and they increase construction costs and add to maintenance problems. In these respects they are no different from natural transverse cracks; however, a significant difference lies in the fact that the joints are present during the entire life of a pavement, whereas, in an unjointed pavement, the transverse cracks develop gradually. In other words, during most of a pavement's life points of structural discontinuity, and therefore structural weakness, are present earlier and in greater numbers in a pavement with joints at close spacing (50 ft and less) than in an unjointed pavement.

From 1933 to 1938 it was our policy to install joints at 30-ft intervals, yet a statewide survey of about 3000 miles of unjointed pavements and pavements with joints 800 to 1000 ft apart showed that the average interval between natural transverse cracks was 28 ft at an age of 13 years. This led one of our district engineers to remark, "When we install joints at close intervals we build the pavement old to begin with." A later survey, which included large mileages of both jointed and unjointed pavements, showed that, on the average, pavements with joints at 30-ft intervals, 2 to 5 years old, had as many transverse cracks and joints as unjointed pavements 12 to 16 years old.

For many years we have been fed a rich diet of joints, seals, fillers and load transfer devices, highly seasoned with unsupported claims for their use —and many have been used. We are not satisfied with the performance of any of them. What with thicker pavements, increasing traffic, insistent demands for heavier loads, higher costs, dwindling dollars, and with a long look into the future, perhaps we need a change of diet. Certainly we need much more sound research.

AUTHORS' CLOSURE

The authors are in full agreement with Russell that many pavement joints constructed in the past have proved to be a major source of difficulty. They also agree that the complete elimination of joints is a highly desirable objective.

There are, on the other hand, reasonable grounds for questioning whether or not joints can be completely omitted without incurring difficulties far more serious than might result if joints were installed—that is, unless we resort to the design of pavement known as "continuously-reinforced" which, in its present state of development, calls for the installation of a much greater quantity of reinforcing steel than usually installed and, in consequence, is relatively costly. In any event, it is the opinion of the authors that in considering this question the following points should be taken into account.

1. Because of the relatively low tensile strength of concrete and its susceptibility to contraction and shrinkage with a lowering in temperature and loss in moisture content, it appears impossible to construct a concrete pavement in such a way that, over an extended distance, it (with reference to the concrete itself) will remain as an unbroken, continuous ribbon. As a matter of fact, the indications are that the maximum attainable length of pavement having true continuity is about 200 ft, which is considerably in excess of that usually attained in actual practice.

2. Any section of concrete pavement constructed as a continuous ribbon exceeding 200 ft in length, and which is exposed to the climatic conditions prevailing throughout most of the United States, is destined to become divided into a series of shorter sections or slabs by transverse cracks occurring at erratic intervals—the actual interval depending upon the influence of a number of factors. The most undesirable condition appears to be the occurrence of cracks at long intervals, in which event there is (a) a correspondingly greater opening of the cracks during cold weather, (b) a loss of the interlocking of the aggregate which might otherwise be sufficient to prevent faulting and (c) a greater infiltration of water and solid materials into the cracks.

3. The ultimate occurrence of transverse cracks in a long section of concrete pavement is not preventable by means of reinforcing steel—in fact, the greater the amount of longitudinal steel the greater the number of cracks. If the conventional amount of steel is installed, the steel passing through a certain number of the cracks will eventually fail, allowing these cracks to open. If, on the other hand, as in the continuously-reinforced design, an unusually large amount of longitudinal steel is installed, the pavement will crack transversely at close intervals. In this case, however, none of the cracks will open to any great extent, at least in terms of recent experimental pavements of this design.

4. The application of prestressing apparently will not eliminate the need for joints, since the length of a prestressed slab has definite limitations from both a practical and economic standpoint. It consequently appears necessary

that there be joints of one kind or another between prestressed slabs. Furthermore, the design of these joints becomes increasingly difficult as the length of slab is increased—bearing in mind that, under normal conditions, the amount of daily and seasonal change in joint width is essentially in direct proportion to the length of slab.

It therefore appears that with the concrete of today, and with all known possible methods of construction, a concrete pavement must of necessity consist of a series of slabs or sections of pavement that are separated from one another by either transverse joints or transverse cracks—it apparently being impossible to simultaneously eliminate both. The problem consequently boils down to the question of which is the least objectionable. This question is, however, complicated by the fact that there are wide cracks and narrow cracks, poor joints and good joints, each type being in a class by itself. We therefore need to exercise care in making comparisons. There also is the important question as to the character and volume of the traffic the pavement is to carry. In the authors' experience, under conditions of heavy truck traffic open cracks and weak load-transfer devices are certain to lead to early failure.

At any rate, under heavy truck traffic, which is steadily increasing on our major highways, the complete elimination of joints appears to permit only one alternative, namely, the construction of continuously-reinforced pavements, which are relatively costly. Furthermore, pending the results of present experiments, and perhaps additional experiments, the long-range serviceability of this design still remains to be determined. These remarks are not intended to cast doubts on the possibilities of this design, but only to point out the present state of affairs, and what the design involves.

Until recently, with a few exceptions, highway engineers have not devoted anywhere near the amount of serious study to the joint problem that it deserves. Actually, most of the development work in connection with load-transfer devices, supporting assemblies and incidental parts appears to have been done by the manufacturers of metal products—as evidenced by the large number of various proprietary devices used and on the market. This being the case, one can well ponder the question of whether or not the failures that have occurred actually constitute reasonable grounds for either surprise or complaint.

In the past, too many road-building agencies have permitted their choice of a joint device to be more influenced by its cost than by its adequacy—the result being the installation of tens of thousands of devices that, to all intents and purposes, were practically useless. If, then, these inadequate devices have failed to fulfill their intended function wherein lies the blame?

The authors concur with Russell that there are still a number of difficult problems associated with joints that have not yet been satisfactorily solved. On the other hand, substantial advances have been made in the design of

load-transfer devices to the extent that the faulting of joints so common to the pavements of the past, and which has constituted one of the most serious forms of joint failure, need hardly be more than a remote possibility in future construction.

It goes without saying that all of the various forms of failure that have occurred in connection with joints and concrete pavements in general have definite causes. Once we take the time and trouble to find out specifically what these causes are, the devising of appropriate means to prevent these failures should not prove too difficult.

Disposition of overflow from large dams creates difficult problems in design and maintenance. The author's experience in this field should prove of interest to the designers of dams.

Effects of High-Velocity Water on Bonneville Dam Concrete*

By R. R. CLARK†

SYNOPSIS

The author compares the progressive erosion which has taken place in the stilling basin of Bonneville Dam with that anticipated in the design, together with the repairs that have been made and the effect on the repairs of the action of high-velocity water.

HYDRAULIC CONDITIONS AFFECTING DESTRUCTION OF CONCRETE

The Bonneville Dam spillway is designed to pass a flow of 1,600,000 cfs, perhaps the largest spillway capacity of record, and it has now been in operation since 1937, a period of 12 years. About 1,000,000 cfs passed this spillway during the high-water period of 1948, producing theoretically about 5,000,000 hp in the stilling basin. Each year flows of more than half this amount occur for a period of several weeks, and lesser flows for periods of greater duration. The forces of these flows are largely absorbed in the stilling basin. An examination at this time of the effects of such flows on the concrete will yield information which will be of a practical nature to designers and will become a guide to the annual maintenance requirements of the structure. Fig. 1 presents general information as to the size of the spillway.

Theoretical hydraulic considerations affecting a structure such as this, as well as any submerged body, may be described as follows. (See Fig. 2 and 3). If in a stream of water flowing with a depth d there is placed a cubical solid with two faces normal to the direction of flow, the water will impinge on the upstream face and be diverted towards the sides and top and around and over the block. If the velocity of flow is low, the water will closely follow around the corners of the block making the sharp directional change with little or no expenditure of energy, as

*Presented at the ACI 46th annual convention, Chicago, Ill., February 21, 1950. Title No. 46-60 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 10, June 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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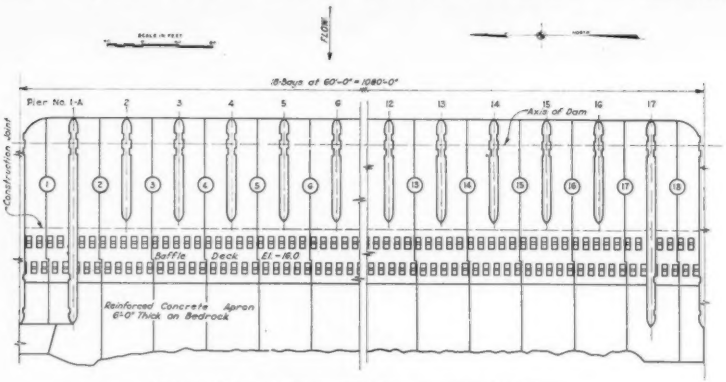

Fig. 1—Plan of Bonneville Dam apron and spillway

evidenced by the development of only small eddy currents. As the velocity of flow is increased, the water cannot follow around the sharp corners of the block due to the inherent momentum of the water particles; there then occurs what is known as separation, *i.e.*, separation of the flow from the block as it progresses past the upstream corners. Such separation is accompanied by the formation, immediately downstream from each corner, of a distinct eddy pocket of reduced pressure. As the velocity of flow is further increased, the pocket enlarges and pressures therein are further reduced. When these pressures have been reduced sufficiently to approach the vapor pressure of the water, the pocket becomes a partial void and cavitation takes place. At this point it should be noted that the formation of cavity pockets is primarily dependent on three factors: velocity of flow of the water, static pressure on the block, and shape of the block. If the static pressure is increased (as by increasing d the depth of flow), it will require even higher velocities of flow to effect cavitation. If, instead of the block being cubical and having sharp upstream corners, the block be streamlined or the corners rounded so that the high velocity flow could more easily follow around the sides, the tendency toward the development of separation (and subsequently of cavitation) would be reduced. Following the development of cavitation, the cavities are swept downstream to regions of higher pressure where they collapse with forces approaching the infinite. If this takes place in the stream away from the block, no damage is done; but if it occurs against the sides of the block, which normally would happen, the almost infinite force of implosion is directed against the material composing the block. Destruction of concrete and steel structures as evidenced by pitting and gouging out of the material will always occur, therefore, just downstream from where cavitation

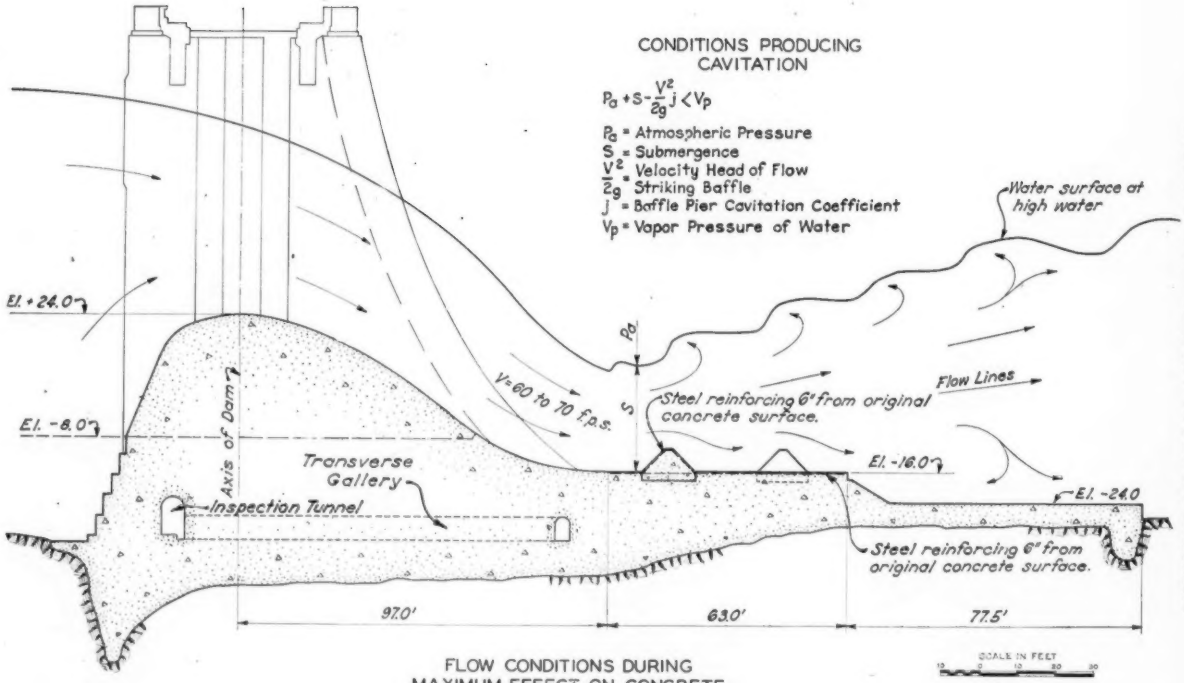

Fig. 2—Typical spillway section, Bonneville Dam

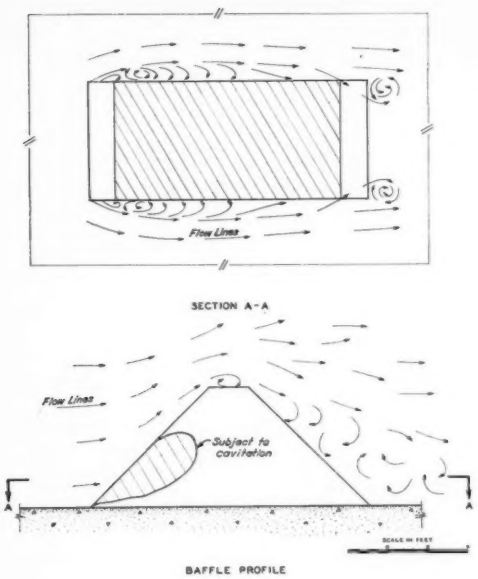

Fig. 3—Baffle flow distribution

takes place and is a direct result of the imploding thereon of the cavity pockets.

A stilling basin is a hydraulic device for converting kinetic energy into potential energy; *i.e.*, converting rapid flow into tranquil flow of greater cross section. This is accomplished by a hydraulic jump accompanied by a tremendous amount of turbulence and transformation of some kinetic energy into heat, raising slightly the temperature of the water. Baffle piers help effect this turbulent change in the state of flow and their use usually results in less costly stilling basin structures.

The above discussion on cavitation applies to Bonneville Dam as follows. The first cavitational factor is the velocity of flow striking the baffle piers. This varies considerably with changes in the spillway gate openings. Although the velocity of flow from beneath the gate is fairly constant at 55 fps, the mass of flowing water is dependent on the degree of gate opening to the extent that by the time it reaches the baffle piers its velocity has been reduced to a value inversely dependent on the mass itself; the greater the mass, the less the reduction of its velocity from the time it issues from beneath the gate until it strikes the baffle piers. The second cavitational factor is the pressure effect on the baffle piers by tailwater submergence. At low tailwater

elevations, and the corresponding lower hydrostatic head, cavitation probably is greater than at high tailwater elevations. It would, therefore, be unwise to operate the spillway with large gate openings during periods of low tailwater. Fortunately this is not likely to occur to any great extent, as the river discharge is low at that time, or the deterioration of the baffle piers would be more severe. The third cavitational factor is the shape of the baffle pier. In the original design of the piers an attempt was made to streamline them as much as possible without losing the efficiency of the piers in creating turbulency in the stilling basin. The upstream and downstream faces of the piers were sloped, but the corners were chamfered instead of rounded.

Thus it can be stated that baffle piers placed in stilling basins and subjected to high velocity flows are almost certain to undergo some erosion by cavitational effects unless the causes of cavitation are properly recognized in the design by providing sufficient tailwater depth over baffle piers, and planning a minimum of operation of the spillway gates at openings which would cause excessively high velocity flow to impinge directly on the baffle piers. A well-balanced design might properly require maintenance or renewal of baffles at calculated intervals.

DESIGN AND CONSTRUCTION

The design and construction of the Bonneville Project was under the direction of the Portland District, Corps of Engineers. Construction of the spillway was accomplished in three major phases. The first step consisted of cofferdamming and erecting the south half of the dam, comprising bays 12 to 17, with the spillway crest between piers temporarily left at an elevation 32 ft below the design crest, to provide slots for river diversion during the subsequent steps. The second step consisted of cofferdamming and erecting the north half of the dam comprising bays 1 to 11, with the spillway crest carried to full height; during this step the entire flow of the river was diverted through the slots provided by the low crests left between the piers placed during first step construction. The third step consisted of progressively cofferdamming each bay of the first step construction and raising the first step spillway crests to design height.

Cement used in the concrete for the spillway consisted of three parts of portland cement of modified composition interground with one part of calcined pozzolanic material. Thus, each barrel of pozzolanic cement used contained only 75 percent of portland cement. While the addition of pozzolanic material produced a small additional compressive strength in the concrete over a long period, early strength was sacrificed for low heat rise in the mass concrete.

Natural river gravel and river sand blended with a terrace sand were used in the concrete mixes. All materials had previously been determined to be of good quality by an extensive testing program.

Mass concrete mixes contained 0.90 to 1.00 barrel of pozzolanic cement per cu yd, the actual portland cement contents being 0.675 to 0.75 barrel per cu yd. Face concrete placed on all surfaces of the spillway subject to overflow contained 1.45 barrels of pozzolanic cement per cu yd, representing 1.087 barrels of portland cement per cu yd. As stated in an article published in the ACI JOURNAL, Jan.-Feb., 1937, *Proc.* V. 33, p. 183, the face concrete mix had the following additional properties: maximum size of aggregate 3 in.; sand to gravel ratio 35 percent; W/C ratio by weight 0.52; slump 2¾ in. at mixer; compressive strength at 7 days 2630 psi; and at 28 days 4600 psi.

That portion of the fishway facilities at the south end of the dam was constructed during the time the river flow was passing through the slots provided in the first step. To carry on the fishway construction, it was necessary to enclose the shore area in a secondary cofferdam. This structure was of earth and rock fill and it required continuing maintenance and repair consisting largely of dumping broken rock on the fill, much of which was subsequently carried downstream through the slots and then upstream again by a strong eddy. Some of the material was undoubtedly washed about on the baffle deck for many months. This portion of the dam was built during winter weather with some subfreezing temperatures, but it is believed that the concrete was not damaged appreciably by freezing.

SURVEYS OF STILLING BASIN CONCRETE

During the third step construction, areas which had been damaged by erosion were discovered and were surveyed by divers during the cofferdam installation. It was believed at that time that observed erosion was due to the impact of heavy jets of water and of solid material impelled by the flowing water and by severe eddy conditions downstream. As a result of this condition it was decided to initiate a complete survey of the piers, decks and baffles.

The dam was completed and the spillway began functioning about March 1, 1938. In 1939 a survey covering the entire area from the spillway to the downstream edge of the baffle deck and from pier 1 training wall to pier 17 training wall was planned and executed as carefully as possible. The diver making the survey was in constant telephonic communication with a plotter and all information was placed on a map as rapidly as the diver's survey progressed. This thorough survey of 1939 has been the basis for comparison of subsequent surveys.

In general, the 1939 survey disclosed that erosion occurred throughout the length of the spillway and in the same general locations on the baffles and baffle deck. The points of severest attrition were on the deck adjacent to the downstream corners of the baffles and on the baffles at their corners and on their vertical sides. The erosion at these points had apparently been caused by cavitation. Other areas of the baffles were recorded as either rough, slightly rough, or smooth; the deck around the baffles other than the areas mentioned above did not show excessive wear with the exception of bays 12 to 17, inclusive, through which diversion had taken place during the second step and part of the third step of construction. In these bays large areas of the baffle deck had been eroded; in most instances the worn areas extended upstream from the upstream end of the baffles to the construction joint between the ogee section and the baffle deck, a distance of approximately 8 ft. These areas extended parallel to the axis of the dam for several feet with a maximum worn area 45 ft long between bays 14 and 15. The maximum depth of erosion in this area was approximately 11 in. The relatively severe attrition in bays 12-17 could be attributed to several causes. Debris from construction operations and wash from the earth-rock cofferdam around the fishway, as previously mentioned, could have accelerated the erosion by grinding action on the baffle deck during the diversion period. However, study and analysis of the problem leaves some question as to whether this was the major cause. In studying the surveys, it was noted that severe wear stopped at the construction joint between the ogee section and the baffle deck, *i.e.*, did not extend upstream from this point. During the diversion period, flow was passed through uncompleted spillway bays with the crest constructed to elevation −8.0 ft. Flow passed over these bays as a broad-crested weir to drop 8 ft farther down onto the baffle deck. It would appear possible that this flow impinged on the eroded area at an angle of approximately 30 degrees from the horizontal and in conjunction with the submerged debris might have caused the excessive wear. The fact that this wear or erosion occurred downstream from the construction joint shows that no cavitation took place underneath the nappe springing off the low bay, otherwise pitting of the ogee section would have been noted just below elevation −8.0 ft. After the crest was completed to elevation +24.0 ft, flow occurred normally down the ogee section and smoothly onto the baffle deck. Pier noses in these same bays also showed the greatest erosion; the low diversion slots and high velocities could similarly have developed cavitation forces causing greater wear on these sections of the piers.

Subsequent surveys were limited to bays 5, 9, 13, 15, 16 and 17, it being the opinion that these selected bays would give sufficient infor-

mation to avoid the necessity of covering the entire area. These surveys were made during low-water periods of 1941, 1943, 1946 and 1948. It is evident from these subsequent surveys* that the wear continues to increase; however, the major portion of all erosion occurred in the two-year period of 1937 to 1939, with the rate of wear decreasing each year thereafter.

Records indicate that up to 1000 parts per million of sand and silt are carried in the Columbia River at Bonneville during annual flood conditions. Particle size of this material is 30-mesh and finer. The silt, so-called, comprises mineral particles rather than soil or vegetative matter. There can be little doubt that this silt-laden water has a scouring effect on the concrete surfaces. This condition has been noted on the ogee crest, roughened deck surfaces, rounded corners of the baffles, and roughness on the upstream sloped face of the baffles. This type of wear has not been excessive, but it is, nevertheless, noticeable.

Bay 5, which is typical of the north half of the dam, was unaffected by the severe conditions existing during construction. (See Fig. 4). It is evident that noses of piers, baffle decks and baffles show progressive wear from the beginning of operation of the spillway. The cavitational pocket-like holes at the downstream corners of the baffles appeared in 1939 and slowly increased in size and depth, although they now seem generally to be stabilized. The wear on the deck progressed more slowly being only a few inches deep after 12 years of service. On the vertical faces of baffles the pitting in localized areas appeared in 1939, two years after being placed in service, and progressed slowly to a more or less stabilized condition. This condition is not uniform, though, as will be seen from the figures showing areas repaired in 1948-1949 in other locations.

In bay 13, through which diversion took place for two years, it was noted that in 1939 the pier ends and deck showed considerable erosion which increased to 1946 and to a lesser degree after that time (See Fig. 5). The effect on baffles was much the same; that is, some wear was noted at the first underwater survey in 1939; it continued to 1946, after which it progressed less rapidly. The downstream row of baffles and the deck in that region show considerable less wear. The baffles were staggered under the assumption that the jet of water would be split, with that portion passing between the upstream baffle striking the downstream baffles directly on the sloping face, but evidently, as indicated in the second paragraph, the spreading of the jet as it encountered the first row tends to break up the remainder of the flow thus reducing its energy and reducing the effect downstream.

*Additional data on the underwater surveys are available from the Institute at cost of reproduction or may be obtained by writing the author direct.

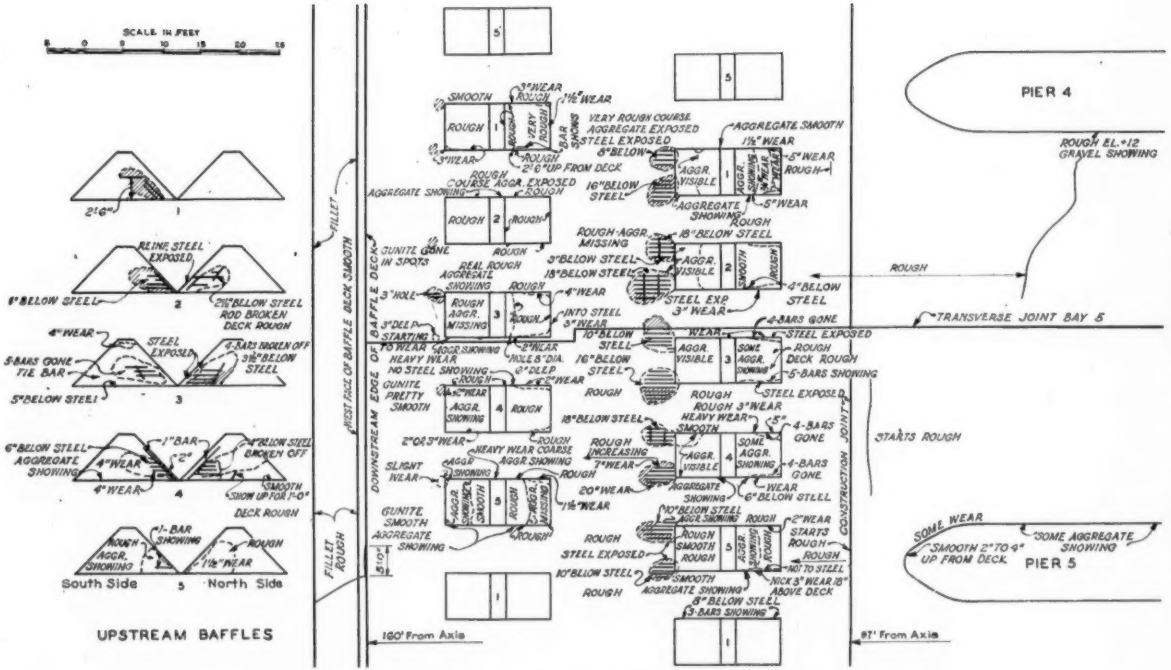

Fig. 4—Condition of baffles, deck and piers of Bay 5 in 1946—No diversion of river during construction

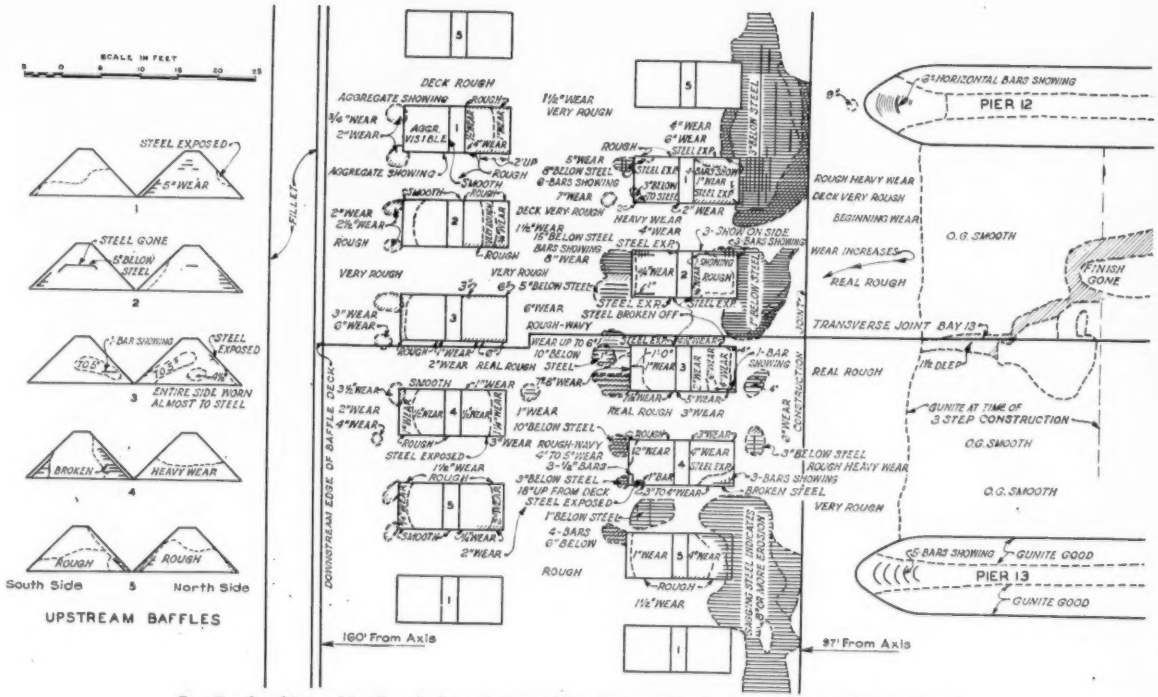

Fig. 5—Condition of baffles, deck and piers of Bay 13 in 1946—River diversion during construction

As indicated earlier, original concrete for all parts of the spillway dam employed portland pozzolan cement. At the time this cement was procured, chemical tests did not include determination of the alkali content; thus the possibility of alkali-aggregate reaction having an effect upon the durability of the concrete was not given consideration. Petrographic studies made within the year from thin sections taken from mass concrete cores indicate little evidence of alkali-aggregate reaction and there is no evidence of deterioration from this cause. There was little evidence of bleeding detected in the specimens examined. It is possible that Type I or Type II cement may have withstood the action of high-velocity water better.

The condition in bays 16 and 17 before repairs (see Fig. 6) was about the same as in bay 13, and after repairs, attrition on the deck did not develop as rapidly as in other cases listed, but the pitting on vertical faces of baffles again occurred showing that the jet of water leaving the upstream corner of baffles creates the cavitation conditions described earlier. It is interesting to note that the near-record spring freshet of 1948 resulted in little appreciable increase in wear over that observed in the 1946 survey (Fig. 7).

Model studies of the Bonneville spillway deck were conducted at Carnegie Institute of Technology prior to 1947 and it was determined that the shape and dimensions of the baffles and deck should be modified by rounding corners and increasing the width 6 in. A survey of damage was made as soon as possible after unwatering each setting of the caisson. Fig. 8 and 9 shows the condition of deck and baffles after ten years of service during which time no repairs were made in this area. Very little difference was evident between bays 15 and 16. These surveys were made to ascertain the actual condition existing, and as a basis for determining the accuracy of information gained by diving

Fig. 6—Condition of downstream end of baffle No. 2, Bay 16, in 1947

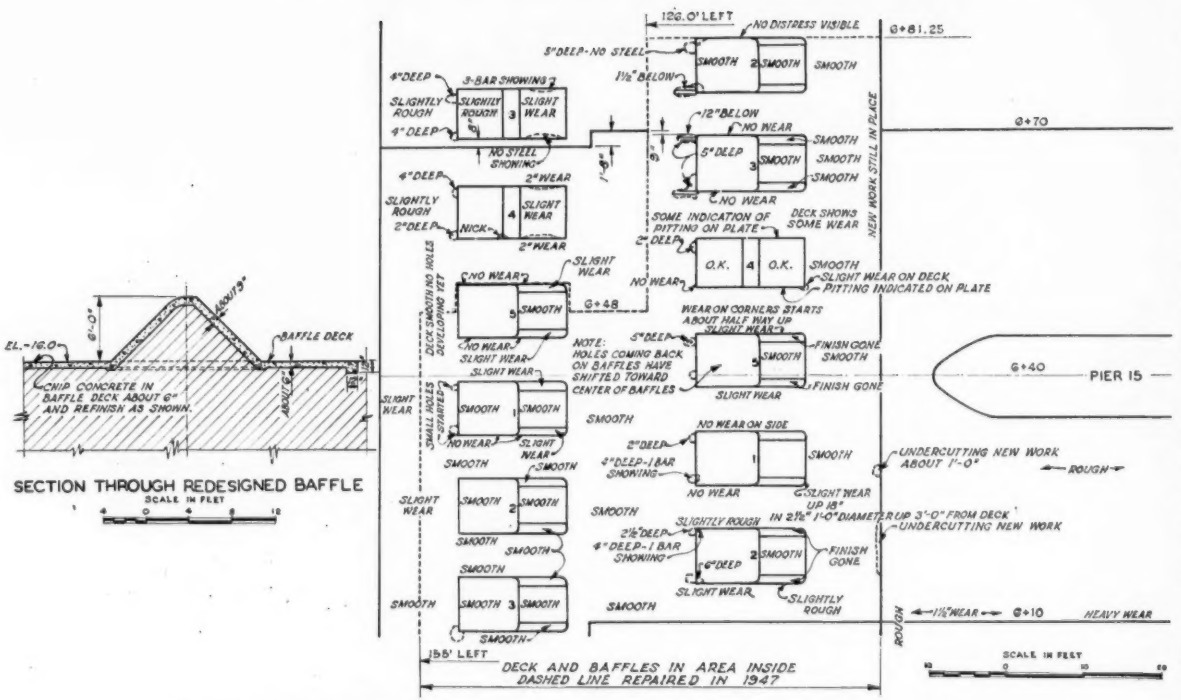

Fig. 7—Condition of baffles, deck and piers of north half of Bay 15 and south half of Bay 16 in 1948

Fig. 8—Condition of upstream side of baffle No. 2, Bay 15, in 1947

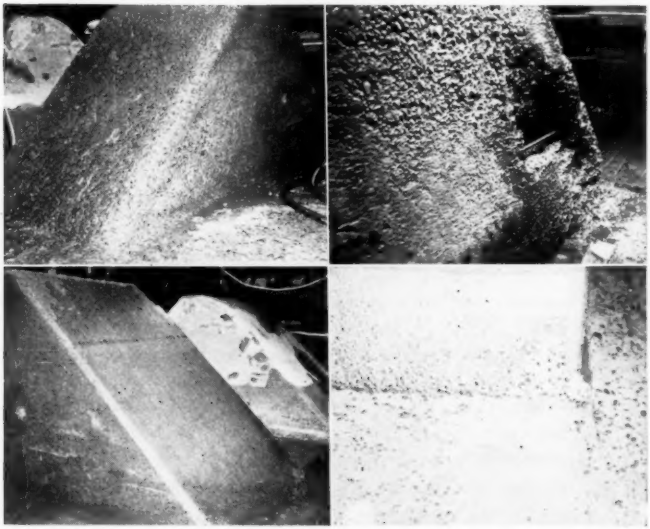

Fig. 9—Condition of baffle No. 4, Bay 15, in 1947. Upper left—Upstream end and side. Upper right—Upstream corner. Lower left—Downstream end and side. Lower right—Downstream end

surveys. Based on this comparison the diving surveys were found to be remarkably accurate as to dimensions and location of damage.

Eight months after the completion of the repair work in bays 15 and 16, and following the near-record flood, another survey was initiated and the effects of attrition were found to be about normal. Special attention was given to the armor-plate baffle in bay 15. The diver reported that evidence of pitting was appearing on the sides of the steel plate.

A recent diving survey made in October, 1949, indicates that the new baffle design, described above, has not been as great an improvement over the old design as tests indicated that it should be. The following changes are evident. Wear along the sides of the baffles has shifted a little farther downstream and wear on the deck at the downstream ends of the baffles seems to have accelerated with deepening generally over the 1948 survey. Wear on the rounded corners of the new baffles is evident in this latest survey, and equals or exceeds that of the original baffles in this respect. Another localized area where cavitation undoubtedly occurred was at the base of each pier adjacent to the gate slot in each bay of the spillway. The crest was also attacked in an area extending outward about 2 ft from the pier base in the same location. This condition developed in the early operation of the dam and was convenient for inspection and repair at each low-water period since the crest of the dam is well above tailwater for several months each year. The areas of attack were small, not more than 1 to 2 ft in size, but were cut rather deeply. After steel plates had been installed, cavitation in six years of operation had cut completely through some of the ½-in. thick plates.

REPAIRS TO SURFACE OF CONCRETE

The first restoration work undertaken on the Bonneville Dam baffles and baffle deck was done in bay 17 and in the south half of bay 16 on the upstream baffles and deck (See Fig. 10). This work was done during the fall of 1940. Baffles in bay 16 were in sufficiently good condition to dispense with repairs. In the repair of baffles and baffle deck, all members were restored to former lines and grades. Since the repair work on the deck extended only to the upstream construction joint an abrupt shoulder was produced, as the area upstream from and beyond the construction joint, while eroded, was outside the limits of the caisson leaving the new work elevated several inches above the eroded concrete deck. It was anticipated that water might possibly undermine the new work at this point and peel it off; however, the 1943 survey showed all the restoration work to be in good condition. A small area on the deck approximately 6 x 9 ft, between baffle No. 3 and the con-

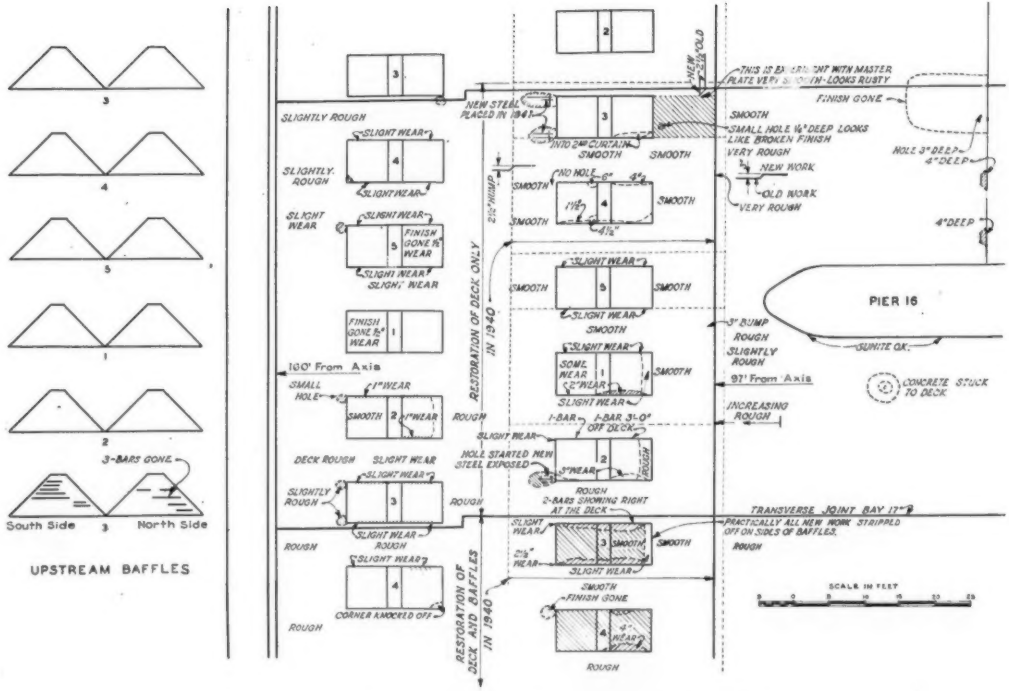

Fig. 10—Results of 1946 underwater survey of baffles, decks and piers

traction joint in bay 16, had been repaired in 1940, by topping the surface with a proprietary metallic admixture to the concrete, basically a ground or pulverized iron. This experimental area has shown exceptional resistance to wear and was in remarkably good shape when examined during the repair season of 1947-1948. This material apparently has considerable merit for such purposes.

Prior to the winter of 1940-1941, equipment had been assembled which consisted of a 30-ton derrick barge (later found to be inadequate for the job), a 185-ton caisson and an auxiliary barge equipped with air compressors, concrete mixer, steam boiler and supplies. The caisson inside dimensions were approximately 32 x 30 ft, enabling it to enclose three baffles at each setting. The over-all height of the caisson was 36 ft with the lower 18 ft containing tanks for floating or sinking.

During the first repair season three settings of the caisson were made on the upstream baffles, and repairs were made consisting of chipping the old concrete down to the original reinforcing steel on the deck and 2 in. below the steel on the baffles. A new curtain of $\frac{5}{8}$-in. round reinforcing steel was placed on the deck, spaced 1 ft each way and midway between the old steel and finished grade. A keyway roughly 1 ft wide was chipped along the construction joint 97 ft downstream from the dam axis with new up- and downstream steel given a 90-degree bend which overlapped the old steel for anchorage in the keyway. Concrete was placed as dry as possible with a final finish being made with a mechanical finishing machine. Steam curing was used during the first 24 to 30 hours to speed up concrete setting as river water temperatures during the working season were from 38 to 39 F.

Work completed during the first season consists of restoring upstream baffles numbers 3 and 4 in bay 17 and restoring approximately 1518 sq ft of baffle deck.

Additional work was planned for the 1941-1942 season; however, the derrick was drafted for war use in the Pacific and no further effort was made to carry on the repair work until the season of 1947-1948. Surveys, however, were made in 1943 and 1946. On reassembling equipment necessary for continuance of the repair work, a new 100-ton derrick barge was acquired and equipped for handling the caisson and other work. This larger barge increased the speed with which repair work could be carried on and overcame hazards encountered with the lighter barge.

The experimental repair work included the protection of one baffle with steel armor plate. This is on upstream baffle number 4, bay 15 (Fig. 7). One existing baffle was removed entirely, being replaced by a new concrete baffle of the same size as the original baffle except that

it was 6 in. wider. The balance of the baffles were chipped and rebuilt according to the new designs which included corners rounded to a 9-in. radius. Floors were chipped following the pouring of baffles and concrete placed thereon as soon as preparations were completed. Plans called for the use of steel forms and Hydron absorptive form liner. On the first baffle repaired the entire steel form was placed, but due to the long sloping face of the baffle the use of heavy reinforcing steel and the Hydron absorptive form liner, it was difficult to vibrate the concrete adequately in the narrow space between steel and form. It was also found that despite meticulous care in placing the absorptive liner, vibration of the concrete caused the form liner to loosen and sag, thereby stopping the flow of concrete. This condition was corrected by using a linoleum cement to hold the Hydron to the steel forms and by splitting the steel form in half. After pouring the first half, placing and anchoring of the upper portion of the steel form required approximately 20 minutes which did not affect the integral nature of the pour. Completion of repairs in the entire area by the revised method was prevented by approaching freshets.

Repairs to local cavitational areas at the base of spillway piers downstream from gate slots at first consisted of refilling with various mixes and cements but it was soon discovered that nothing but metal would withstand the severity of these forces. In 1941, ½-in. steel plates were applied to the face of the piers, welded to slot armoring and thoroughly anchored to the piers. A number of these plates were torn off, and, in 1945, 1-in. thick plates were used at some piers to replace the thinner metal with apparently satisfactory results. In designing the McNary Dam piers where similar conditions exist, it was determined by hydraulic model tests that a slight change in the shape of the pier face in this area diminishes negative pressures and, it is expected will eliminate the trouble.

CONCLUSIONS AND PLAN FOR CONTINUING MAINTENANCE

It is evident from the preceding data that cavitation by high-velocity water as well as abrasion is involved in the condition of the Bonneville spillway and stilling basin. The greater surface damage in the south half of the stilling basin is, however, undoubtedly due in part to the heavy material carried by the water during construction. Cavitation also accounts for the condition on piers adjacent to gate slots, the vertical sides of some baffles, and probably the rounded pits on the baffle deck at the downstream corners of many of the baffles. Much more could be said about some of these conditions. For example, some of the steel plates applied to the cavitational area adjacent to the gate slots were ruptured horizontally, and in some cases about one-half of

the plate disappeared. An apparent explanation of this is that the dynamic forces produced by rapidly changing pressures on the face of the plates continuously for many months produced fatigue in the metal which then failed in this unusual manner. It is often impossible to draw a sharp line of demarkation between erosion, abrasion and cavitation, and the conditions at Bonneville seem to exemplify this statement. At the time the stilling basin was designed it was expected that the baffles at least would require renewal at about 15-year intervals. The conditions that have developed would seem to justify this assumption as being reasonably conservative.

The plan for continued maintenance, as now anticipated, includes repairs to deck and baffles progressively from the south end to the north end of the stilling basin, thus first renewing all parts not already repaired in the area of construction diversion. After this, the same plan of renewal will continue in the north half. Type II cement will be used for all repair work. Special methods of construction will probably be adopted experimentally, such as the prepacked method for baffle construction, and the use of a metallic admixture in the deck finish. The advisability of using reinforcing steel in both baffles and deck in the future is questioned. It is understandable that a designing engineer has an inherent belief that reinforced concrete is more durable than plain concrete, and that for an installation such as this where repairs are difficult, an excess of steel might easily be specified just for good measure. The author has been exposed to about 15 years of design and hydraulic experiments on a rather large scale since this stilling basin was designed, and now believes that baffles such as those at Bonneville should preferably be unreinforced, but with heavy dowels connecting the interior of baffles with the mass below. One reason for this belief is that steel placed within a few inches of the surface sometimes becomes exposed in a short time, after which heavy dynamic forces that exist in the stilling basin set up continuous intense vibrations in the steel which undoubtedly fracture and deteriorate the concrete much more than if no steel had been used. The same can be said for reinforcing steel placed 6 in. below the surface of the deck or floor of the stilling basin. The baffles were placed in recesses provided in the deck and in the future at least some will be entirely removed and built without reinforcement. Laboratory experiments by the Bureau of Reclamation prove that pitting of plain concrete is greatly increased as the water-cement ratio is increased. Concrete of the lowest possible water-cement ratio will therefore be used.

Efforts will be made to reduce pitting on baffle sides by changing the shape. Model studies by the Corps of Engineers in developing designs for baffles at the Blue Stone Dam resulted in curved sides shaped

to follow the nappe of a jet issuing from a rectangular orifice. From this the curved side was established. To follow this plan at Bonneville would result in a considerable increase in the width at the downstream end of the baffllle, thereby restricting the flow passage between the baffles and possibly disturbing the over-all hydraulic conditions in the stilling basin; however, in future repair work this principle will be adopted to a limited extent with the exact shape of side being determined by model experiment; in other words, attempting to fill in with concrete the areas along sides of baffles shown in Fig. 3 as low pressure areas. The results of such changes can be observed in the prototype before many of the baffles have been rebuilt.

Underwater surveys will continue each year to closely observe any unexpected conditions that may develop and as a guide to the most desirable places and methods of carrying out annual maintenance.

An improved type of caisson is under consideration which will be larger in plan, with a view to covering the areas at pier ends and on the crest adjacent to the deck, thus improving the facilities for maintenance, and possibly reducing the cost.

A review of recent developments in the use of precast concrete.

Precast Reinforced Concrete Structures*

By C. D. WAILES, Jr.†

SYNOPSIS

Several examples of the use of precast concrete in industrial, commercial and residential structures are described. Special design considerations, manufacturing methods, erection techniques and the inspection and testing of precast members are discussed.

INTRODUCTION

Interesting and novel things are being done with precast concrete in many parts of the world. In recent years the technical literature has included good coverage of the developments and uses of this growing type of construction. Accounts range from theoretical, academic studies in research to completed methods by which sound and satisfactory structures have been constructed.

It is not the purpose of this paper to survey these many and varied undertakings, but rather to focus attention on the practical application of precast construction in this country.

The art of casting into forms or molds a plastic or fluid to be later removed in a firm, stable shape is as old as the history of man. Portland cement concrete has been cast into various molds to form architectural and engineering shapes since its early development. Precasting, then, preceded "monolithic" or "cast-in-place" construction so generally used today.

Manufactured items such as pipe, concrete masonry units, tile and cast stone are, of course, precast, for rarely are they cast in their final end-use position or location. These are generally referred to as "concrete products" as differentiated from precast, reinforced elements of buildings and structures such as beams, slabs, columns, ribs, shells, wall and floor assemblies and the larger, reinforced architectural concrete elements.

*Presented at the ACI 46th annual convention, Chicago, Ill., February 21, 1950. Title No. 46-61 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 10, June 1950, *Proceedings* V. 46. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1950. Address 18263 W. McNichols Rd., Detroit 19, Mich.

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In view of the many desirable features of cast-in-place concrete construction, we may well ask, "Why precast?" Surely cast-in-place construction offers the ideal in carrying out the valuable principles of continuity so generally used in design today. Nothing is more interesting nor impressive than the teeming activity of carpenters, mechanics, iron-workers and concreting crews as they form, place, cast and strip a huge monolith of concrete, giving both structural integrity and lasting, pleasing architectural expression.

Why, then, the increased interest in recent years in, and the use of, precast methods?

The reasons seem to be (1) economic advantages—saving of time and invested capital, (2) architectural or engineering advantages and (3) the ever present desire to change—to develop, to create something new. Saving of time and invested capital—economic advantages—this is the final test which determines whether the method or system of construction is to survive. The great majority of precast "systems" follow a path of invention, model stage, one or two tryouts, then into discard as far as practical and commercially useable construction is concerned. They satisfied only reason number three.

APPLICATION OF PRECASTING TECHNIQUE

It is interesting to appraise briefly some of the structures which appear to satisfy reasons one or two and in many cases both one and two.

Bridges

A number of small bridge decks of approximately 24-ft span are being constructed by the Pennsylvania Highway Department. Here the reasons for precast construction are evident. Simple, easily handled channel shaped elements are precast under ideal factory conditions and are transported by truck to job site. Installation upon existing abutments is but a matter of three hours. Costly job forming, weather problems, and serious traffic interruptions are eliminated and an estimated 25 percent saving over conventional cast-in-place construction is reported. Many uses of concrete molds combined with vacuum processing contributed to economy and quality. Simple span design combined with preshrinkage resulted in minimizing flexural stresses in existing abutments.

Precast beams, on which a reinforced concrete slab is placed, are being used also to speed construction of bridges (Fig. 1).

Warehouses

The two large Navy Department storehouses* constructed at

*Amirikian, Arsham, "Precast Concrete Storehouses," ACI JOURNAL, June 1947, *Proc.* V. 43, pp. 1097-1116 and Corbetta, Louis P., "Precast Concrete Warehouse Construction," ACI JOURNAL, June 1947, *Proc.* V. 43, pp. 1117-1214.

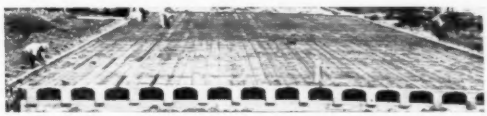

Fig. 1—Precast T-beams in place in a Texas bridge

Mechanicsburg, Pa., illustrate the excellent results obtainable by precast methods when engineering design is directed to the creation of elements of minimum section and these sections are assembled and connected in a manner to develop full continuity.

Some 4000 precast elements were cast in approximately 100 concrete molds indicating about 40 uses per mold as an average. Hollow sections cast in half pieces later to be assembled into final shape resulted in a minimum use of material and a correspondingly lighter structure. The use of higher strength concrete and vacuum lifting mats facilitated production and early handling. Erection was orderly and involved a repetition of sequences which became more efficient as the job progressed. Welding of reinforcing steel combined with cast-in-place "jointing" concrete accomplished continuity in a most desirable manner. Saving of both time and money over other conventional methods proved the soundness of the method.

Marine structures

Illustrative of marine structures of precast concrete is the 26 ft wide, 3800 ft causeway at Point Richmond, Calif. This structure is designed for H20 highway loading and involves the use of 18 in. x 18 in. x 90 ft precast piling, precast forms for the transverse pile caps, and precast deck slabs 6 ft 7 in. x 20 ft of closed end channel section. Members were made in a central casting yard. The precast concrete forms for the cast-in-place caps were received at the job with reinforcing steel cage in place. Formwork over water was minimized and no exposed bolting was needed for connections. Structural soundness combined with speed and economy were obtained by the application of precasting methods.

Housing

In the field of mass dwelling construction precast methods (Fig. 3) have received considerable attention and numerous large scale projects have been completed. Satisfactory castings with windows, frames, utilities and inserts are not difficult, however, some of the existing work shows lack of attention to joint details, inaccuracies in casting and not too well worked out erection procedures. Generally, however, reasonable economy has been obtained even when compared with other types of construction considered to be economical.

Seventy reuses of forms in making 6800 precast floor panels for United Nations Housing in New York resulted in economy and speed of con-

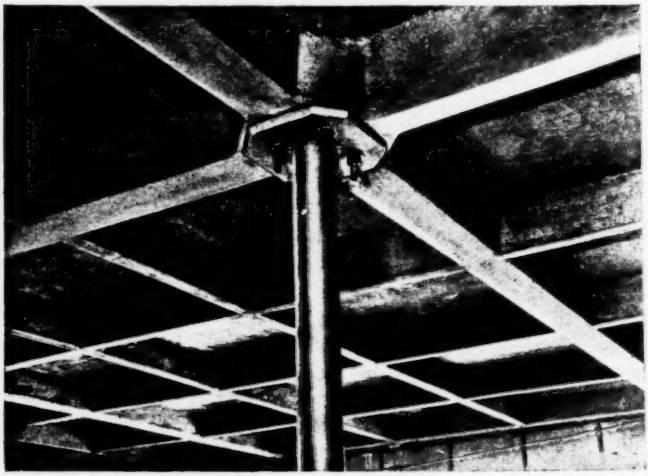

Fig. 2—Precast floor slabs in place on steel columns in United Nations Housing Project, New York

struction. Thin ribbed slabs, averaging 2½ in. of concrete for panels up to 16 x 16 ft results in lighter weight and economy of materials (Fig. 2).

Here, again, concrete molds were used to assure precise castings and vacuum processing and lifting were used to produce high quality concrete and to facilitate handling of the panels. The use of dowels in grouted holes, and overlapping of slab mesh in grouted slab joints provided rigidity of anchorage without resorting to exposed bolting of any kind.

For architectural expression large precast slabs as decorative facings for buildings and other structures have been used with outstanding success. The precast slabs, generally 2 to 3 in. thick, are cast with exposed aggregate textures or tooled surfaces to give the color and character desired. Of large area, up to 100 sq ft, they have been used as a veneer or as forms against which the concrete wall is cast.

This type of facing material offers economy when compared to natural stones, cellular terracotta, etc. and, being of extremely high quality, provides excellent weathering and durability. Recent examples are the Prudential Building, Los Angeles; Pacific Gas and Electric Building, San Francisco; David W. Taylor Model Basin, U. S. Navy, Carderock, Md., and Waterman Steamship Building, Mobile, Ala.

Industrial buildings

Thirteen acres of precast vacuum processed slabs roof the new pottery plant of American Standard Corp. at Torrence, Calif. (Fig. 4). Light

Fig. 3—Setting precast roof slabs to be bolted into inserts in top of wall

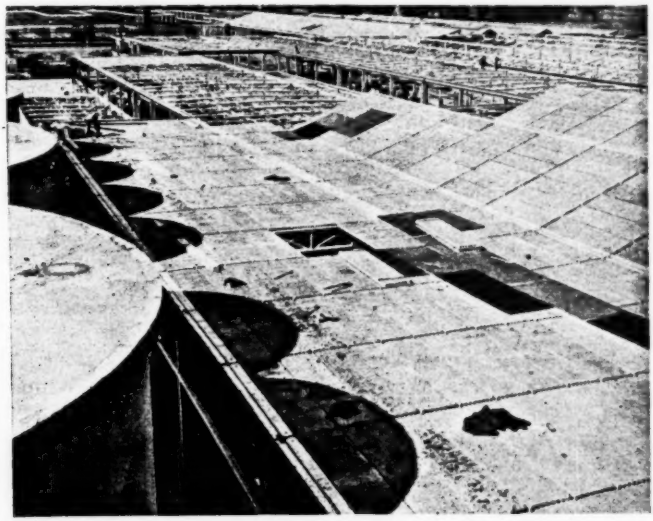

Fig. 4—Hollow precast slabs being placed on the 500,000 sq ft roof of an industrial building

steel trusses spaced 10 ft on center span the 40 ft between bents and the precast slabs span the 10 ft between trusses. Plant manufactured lightweight, cored slabs provide insulative roof and a smooth light colored ceiling. Nelson steel studs are welded to the steel frame at each corner of the slab and all joints between and at ends of slabs are grouted. By this method the roof slab assumes sufficiently reasonable monolithic properties to act as a diaphragm for distribution of horizontal forces within the bent areas. A slab dead load of only 20 psf and cold drawn wire reinforcing of approximately 0.7 psf attest to economy of design. To form and cast in place this roof slab on various slopes 16 to 30 ft above the floor would be far more expensive and might have eliminated concrete in favor of some more economical roofing construction.

A recent example of the application of precast methods to various portions of a large industrial project is a new copper refinery at Garfield, Utah. Fourteen hundred rectangular precast electrolytic vats rest upon precast bents and 500,000 sq ft of precast lightweight channel slabs cover the roof. The contractor elected to do all precasting at the job site and 2380 rigid frame bents were precast. When 16 hours old they were stockpiled, using vacuum lifters.

The concrete vats, approximately 5 ft 5 in. x 4 ft 4 in. x 14 ft 3 in., are cast bottom up against forms lined for vacuum processing. After casting, processing continues under a shroud with steam for 40 minutes; after 1 hour and 20 minutes all forms are stripped and steam curing continues for 6 hours. Channel roof slabs are of lightweight concrete cast in assembly line production into steel forms, vacuum processed, steam cured, stripped and stockpiled at the rate of 240 per 6 hour production.

Forming and casting in place the roof slab high above the floor level or casting in place the hundreds of rigid frame bents would have been far more expensive than was precasting.

Tilt-up construction

The high cost of forming during the past five years has brought forth the economy in precasting walls of industrial and certain commercial buildings. The practice of casting wall panels in a horizontal position on the concrete floor of the building and later standing or tilting them into position has gained much headway and some good examples of this practice have been observed. Some of the designs involve a half pilaster along each vertical edge so that when erected and joined the pilaster is created (Fig. 5). The elements of the pilaster stiffen the slab for erection and handling.

Fig. 5—Lifting 16-ton wall slab

Other uses

Reference to practical precast methods would, of course, not be complete without mention of other elements of construction such as joists, beams, hollow slabs, stairs, lintels and many other types and kinds which have become standard products of manufacturing firms and which have been used with economic and structural advantages for years (Fig. 6).

These are but a few of many examples of practical usage of precast methods in various types and kinds of structures where soundness of design, construction procedures and invested capital were all served to advantage.

DESIGN CONSIDERATIONS

To obtain optimum advantages from precasting, the designers, both architectural and structural, must know something of the techniques and procedures involved in manufacturing and erection.

Fig. 6—Hollow Flexicore roof slabs placed on a cast-in-place concrete frame

Naturally, repetition of use of molds offers economy—where many castings are to be made from a mold more care may be given the construction of the mold itself. Modular layouts are advantageous. Consideration should be given to the fact that fitups are never to exact dimensions. Slight variations from the theoretical size must be provided for in designing architectural facing slabs or large areas of structural floor or roof slabs. Generally the elements should be made slightly less than the modular dimensions established.

The use of continuity in structural design so desirable in monolithic construction is obtainable in precast frames and larger members. However, it may be, and generally is, to advantage to consider only single span design for joists, slabs and the smaller elements.

Certain structural advantages may be taken of the higher strength concrete obtainable in precast work. Placement of steel and concrete and adequate curing may be more fully relied upon. There are sensible limits, however, and caution should be exercised to assure adequate coverage of steel, location of anchors and inserts and provision for warpage due to shrinkage and unsymmetrical sections, particularly when castings are of lightweight concrete.

The welding of field connections should be encouraged. Reinforcing steel splices, face plates embedded in the concrete to be welded to steel members, stud dowels, etc. may be easily welded without damaging the adjacent concrete. When such methods are used exact fitups are unnecessary.

Bolted connections, if used, should be carefully detailed. The spectacle of sizable concrete members bolted together by clusters of bolts near edges and ends of members is not too reassuring when we think of some of the less desirable physical properties of concrete.

Concrete, Gunite or grout joints may be relied upon where compression and shearing stresses are involved. The technique of grouting to minimize shrinkage cracking is well-known.

Careful attention must be given to deflection of shallow or long span members especially when designed in lightweight concrete. Sometimes, in unsymmetrical sections, drying shrinkage produces a deflection which is additive to that produced under load.

It is not reasonable to assume that many small precast elements can be assembled into a floor system sufficiently level and smooth to receive, directly, thin finishes similar to asphalt tile. Where such finishes are used an underlayment should be employed. Asphaltic mixtures with sand and cement may be spread and trowelled ¼ to ½ in. thick or portland cement and sand topping may be applied ¾ in. thick to a properly prepared surface.

MANUFACTURING

Concrete construction basically resolves itself into the handling of weight and placing it in its final position. Precast construction reduces this weight considerably by eliminating much falsework and by the use of lighter precured and dried members. The handling of this weight is one of the considerations in deciding whether the work is to be done in a plant or at the job site.

In general, job site precasting will prove more economical for the larger, heavier elements which may be cast close to their final use position in the structure, so there will be a minimum of rehandling (Fig. 7). Weather conditions, space requirements and higher job wage scales, however, may offset other job site advantages. Where sections are too large for satisfactory truck handling it is, of course, essential that they be job cast.

After shop drawings are made and approved, job tickets are prepared giving all dimensions and details for each item to be manufactured—quantities are summarized and special conditions are noted.

Reinforcing steel assemblies are made on a jig and tack welded. Where small rods or cold drawn wire are used in mat assembly, prefabricating in specialty shops by use of gang spot welders is advantageous, as the coils are straightened and made up into mats in a continuous operation and waste is minimized. Where possible, inserts, anchors, faceplates, etc. are welded to the reinforcing steel. Position-

Fig. 7—Precast side wall panels. Marginal beams are half of pilaster, horizontal beam is bond beam at plate line.

ing of steel by wire chairs or wire indexes tack welded to the mats is desirable. Assemblies are tagged and stockpiled or hung on racks adjacent to their use position.

Forming and casting methods are determined by the type and quantity of the product to be produced. Small slabs, beams, joists and relatively small items are generally produced on a vibrating machine in steel forms. Manufacturers of catalogued products such as joists, channel slabs, and planking, of course, have completely developed production lines. These products may be purchased more inexpensively than the contractor could produce them. Methods of placing and compacting concrete in these small and sometimes thin sections generally involve vibration by table or form type vibrators.

Handling is on pallets or bottom forms until stripping position is reached. Semi-extrusion methods are used on some such products, generally after vacuum processing, so all forms except pallet form are removed or the product is pushed from the forms.

Handling of these products must be such that distortion is prevented. Steam curing is generally used for 8 to 12 hours. In stockpiling these products, sticking or blocking should be arranged so that the product is held properly for drying and shrinking (Fig. 8).

Larger sections, and those of special design, are generally formed by wood or concrete molds. Where sufficient quantity is involved, concrete molds are preferable. The procedure is to remove the product from the

Fig. 8—Method of storing large architectural facing slabs

molds instead of removing forming from the product. Concrete molds are first cast and rubbed to a smooth surface condition. Adequate draw is provided and for large castings provision is sometimes made for compressed air or water to be injected between the product underface and the mold as an aid in parting the casting from the mold. Form coatings or parting agents are many and varied, with stearates probably predominating.

Low slump concretes, placed by high frequency internal vibration, or plastic mixtures, later vacuum processed, are both in general use. These methods combined with thorough mixing and scientific proportioning generally produce concretes of from 4000 to 8000 psi. Such concretes are sufficiently strong within the first 24 hours to be removed from the molds. Both methods are effective with either lightweight concrete or conventional rock and sand mixtures. Strengths of 2500 to 5000 psi are being obtained from concretes weighing less than 100 lb per cu ft.

Flat slabs and architectural facing slabs generally are cast on concrete or other specially prepared casting tables which may be maintained in a true plane.

Overhead cranes and fork lift trucks are ideal for stripping and handling large products. Hoist speed should be slow at first as the casting is being parted from the mold. Vacuum lifting mats are desirable and speed handling because they are easy to attach and release—simply by opening and closing the valve. The lifting mats can be designed to prevent harmful stresses in products moved at an early age.

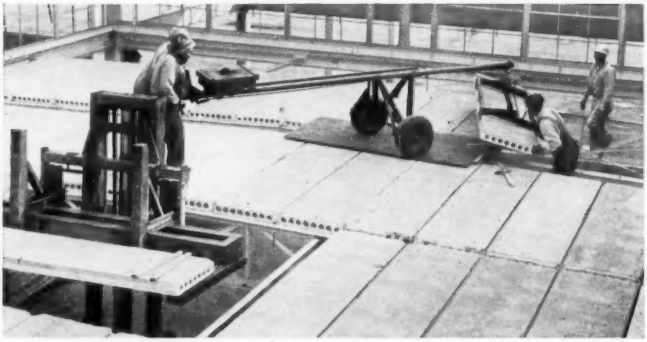

Fig. 9—Slabs handled by jinnywink with extendable boom and counterweight

ERECTION AND INSTALLATION

Where deliveries can be coordinated with erection and installation schedules, loads can be made up on the delivery trucks to arrive in the order needed. Erection sequences, of course, are determined in advance. Crane operators, riggers and welders operate about as they do for structural steel erection.

Small slabs weighing up to about 1000 lb can be conveniently handled and placed by dollies or two-wheeled jinnywinks with counterweights and extendable boom (Fig. 9). Welding of face plates, Nelson stud welding or bolting follows immediately upon setting—grouting follows as suitable areas are developed. Slabs having an area of 25 sq ft each have been placed at a rate of 5000 to 8000 sq ft per day.

Large sections must be handled by crane. If crane equipment is required, equipment and crew time is about the same for a 15-ton section as for a two-ton section. Larger sections therefore are more economical once it is decided to use large handling equipment (Fig. 10 and 11).

Where large slabs are precast and erected for walls, an excellent method for achieving monolithic construction through columns and around corners is obtained by casting the columns or pilasters as closure elements after the walls are erected. Forms lined for vacuum processing may be used and removed shortly after processing. The use of the vacuum process results in a good bond between the precast and the cast-in-place concrete and eliminates, to a large extent, shrinkage problems at these connections.

INSPECTION AND TESTING

In monolithic construction the members generally are designed to

Fig. 10—Placing canopy on precast columns

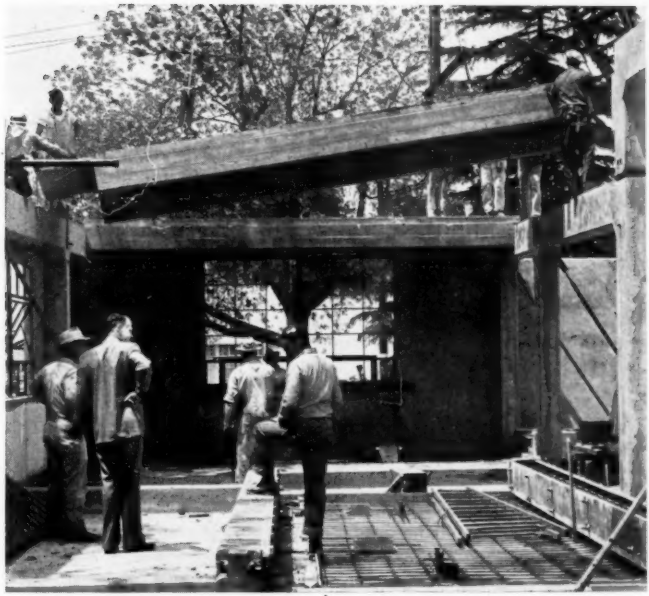

Fig. 11—Setting precast roof slab and beam unit

permit simplicity of forming, ease of concrete and steel placement and compliance with regulations which establish certain practical minimum dimensions. In creating designs for precast members the tendency is to use minimum sections of structural adequacy leaving the "matter of method of accomplishment" up to the contractor or fabricator.

Considerable handling of the smaller members through conveying methods, curing kilns, and form removal indicates reasonable inspection is needed to prevent the use of methods likely to be injurious.

Thin sections require accurate placement of reinforcement and good compaction of concrete and, of course, adequate curing.

Where units are made under special vibration or pressure conditions it is difficult to obtain concrete test samples which have the same processing as the concrete production. The use of standard cylinder tests would not always indicate even approximately accurate results. Some sections are so thin that coring is not very practical and the effect of minor defects, voids, etc., in such small samples affect the results to a considerable extent. Small briquets or cubes may be cast but cannot always be treated consistent with the product manufacture. It would appear that occasional physical tests of the full size member are needed for proper evaluation where small control tests are not representative.

Where lightweight concrete is involved, tests sometimes include the weight per cu ft, thermal or acoustical properties and absorption as well as compression, rupture and modulus of elasticity.

Drying shrinkage and warpage are definitely to be reckoned with, especially with lightweight concrete and unsymmetrical sections. Combinations of these phenomena with a relatively low modulus of elasticity concrete will result in excessive vertical deflection of horizontal members unless properly anticipated and allowed for in manufacture.

The use of higher strength concrete in combinations with prestressed steel of high yield point makes possible economical, lightweight sections but again requires adequate plant inspection and testing procedures to prevent its misuse or misrepresentation.

Handling of large or unsymmetrical members, of course, requires proper care and technique, otherwise excessive stresses may be set up and cracking or bond failures, etc., may occur and yet not be noticeable at the time.

Proper job inspection should include careful attention to the erection and handling procedure, as well as to final setting of these members for proper bearing, grouting and connection alignment. If these are neglected, areas of excessive stresses may be set up which may cause trouble as the building or structure later adjusts itself to settlement, loading and temperature conditions.

SUMMARY

During and since the war precast construction has expanded rapidly to include construction of trestles, bridges, marine structures and many types of buildings on a major scale. It certainly can be conservatively stated that precast construction is now an important method in use by the construction industry.

The architect or engineer, in creating his design, can now consider the use of certain precast sections as marketed and catalogued, as has been his custom with steel and other products. His design skill, however, is not limited to the standard manufactured sections. He has much latitude for original design and may create his structural and architectural elements to best advantage for his particular project.

The use of precast construction is growing rapidly. As its use expands, and its possibilities appeal to the creative talents of the designers, more latitude with design, production and construction will be taken. It has proved an economical and structurally sound method and, therefore, if its continued use and growth is accompanied by practical safeguards, we may expect precast concrete construction to play a major role in the future.

A summary of the more common prestressing methods and their application.

Prestressed Concrete Construction Procedures*

By THOR GERMUNDSSON†

SYNOPSIS

In reviewing the construction procedures used in the production of prestressed concrete, the methods of prestressing are grouped under several simple classifications. A brief outline of these methods is presented, and typical prestressing procedures are described and illustrated.

INTRODUCTION

The first patent on prestressed concrete was issued in 1888 to P. H. Jackson of San Francisco. During subsequent years many other patents were granted, but R. H. Dill of Alexandria, Neb., in 1923-25, appears to have been the first to suggest the procedure of what is now known as "post-tensioning." While some of the first steps may be said to have been taken in this country, the initiative thereafter went to Europe where prestressed concrete was taken up by such prominent engineers as Freyssinet, Magnel and Hoyer.

The basic idea of prestressed concrete is to eliminate tensile stresses by superimposing compressive stresses. This is done by stretching the reinforcement. That much was known more than 60 years ago. The idea was tried out under various conditions, but with little or no success. The neutralizing compressive stresses could be induced but they could not be maintained indefinitely by ordinary bar reinforcement. About 20 years ago Freyssinet in France realized what the trouble was, and when he induced the neutralizing stresses by stretching high strength steel wires, the main obstacle was overcome.

During the last 10 years, especially, prestressed concrete has flourished in Europe. Scarcity and high cost of materials have doubtless contributed to its spectacular progress abroad. Another contributing reason may be the European method of letting contracts, whereby the contractor makes his own design and reaps part of the benefit for

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introducing new, economical types of construction. At any rate, prestressed concrete has grown in stature abroad and has there reached full maturity. It is far beyond the experimental stages and is now an important type of construction.

With the construction of the Walnut Lane Bridge in Philadelphia, prestressed structural concrete has now come back to America. Nothing like it has ever been built here before. The Walnut Lane Bridge is the forerunner, it is believed, of what will be common practice in the future in this country. When American construction talent is brought to bear on the problem of how best to prestress concrete, important new contributions may well be added to the already well-established procedures. In the hope of stimulating such domestic developments, a brief review is presented of the various procedures generally accepted abroad.

OUTLINE OF CONSTRUCTION PROCEDURES

In prestressed concrete preliminary compression stresses are induced in the concrete by means of steel wires. When the wires are stretched *before* the concrete sets, or before it is placed, the wires are said to be pre-tensioned. Post-tensioning describes the condition produced by wires stretched *after* the concrete has hardened.

For each of these two main types of prestressing, a differentiation is made in Fig. 1 between circular and straight structures. Pre-tensioned pipes and caissons have been made according to procedures suggested

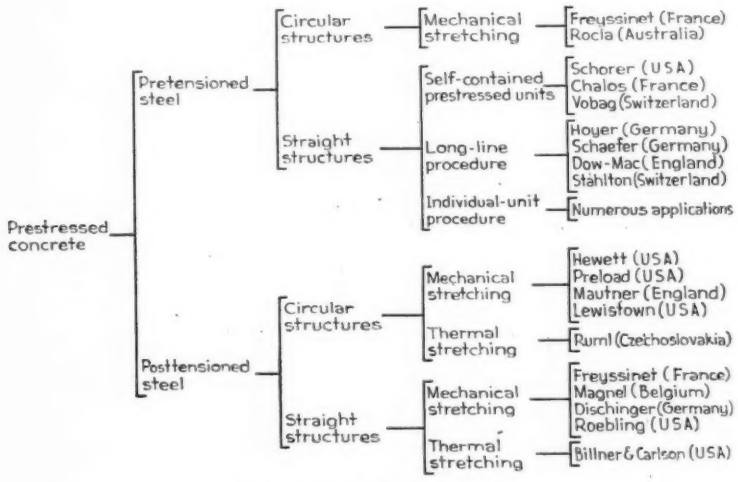

Fig. 1—Types of prestressed concrete

by Freyssinet[1] and by the Australian firm Rocla.[2] In the latter construction, circular reinforcement is embedded in concrete which immediately after having been placed is subjected to high pressure applied to the inside surface of the circular wall. When the concrete hardens the reinforcement remains stretched, and when the inside pressure is relieved, the concrete becomes compressed.

Pre-tensioned straight structures may be made with self-contained prestressed units, of which three types are listed in the chart. Schorer's[3] and Vobag's[4] types will be discussed. Chalo's is similar to Schorer's.

The greater portion of pre-tensioned structures are made in factories especially equipped to make prestressed units. They may be made either on long stressing beds, the units being laid end to end, or the units may be made individually. One type of long-line process is called Dow-Mac.[5] Hoyer's[6] is similar to and preceded Dow-Mac. Schaefer units are made in long integral lengths and then sawed into the lengths needed. Individual units are made in many instances and by various procedures. The units made in Orleans, France, which are typical of this procedure, will be discussed later.

Post-tensioned circular structures such as tanks and pipes may be precompressed by means of rings or bands of steel wrapped around the surface of the hardened concrete. Hewett[7] did this by means of ordinary bar reinforcement provided with turnbuckles. Mautner[8] used high-strength steel wrapped around precast concrete units. Jacks were inserted in the wall and when extended stretched the wire. The openings left for the jacks were then closed so that the compression was maintained in the concrete wall. The Preload[9] type of tank construction is a well-known American procedure. Pipe made in Czechoslovakia by the Ruml method are wrapped with heated wire. On cooling off, the wires become stressed and the concrete compressed.

In straight structures, thermal stretching has been used by Billner and Carlson[10] who employ electrical heating. Practically all other stretching in this group is done by hydraulic jacks. The outstanding methods are those developed by Freyssinet and Magnel.

SELF-CONTAINED PRESTRESSED UNITS

A prestressed reinforcing unit developed by Herman Schorer[3] works on a principle entirely different from any other described here. As illustrated in Fig. 2, two groups of wires, braided in opposite directions, form a cage around a center rod. The wires are stretched by a jack bearing against the center rod which serves as a temporary compression member. The wires and the rod are then interconnected and the jack is removed. The stresses in the rod and wires are preserved within the

Fig. 2—Schorer prestressed reinforcing unit

unit which may be handled, shipped and placed almost like any ordinary reinforcing bar. When placed in a structure the ends of the units are not embedded. After the concrete has gained sufficient strength, the end locking devices are unfastened, which releases the stress on the center rod and transfers it to the surrounding concrete. The center rod is removed and shipped back to the fabricating shop with the end locking devices. These prestressed units have worked out well in experimental installations.

To provide sufficient vertical clearance for electrification of a British railroad, a number of overhead bridge decks are at present being replaced.[11] The composite construction illustrated in Fig. 3 was adopted after considerable study. Conventional reinforced concrete was eliminated in view of the limited construction depth available. Bids were taken two ways, one for the structure in Fig. 3 with prestressed precast concrete joists combined with cast-in-place concrete and another for encased steel beams. The latter was 50 percent higher based on the deck slab only. The wires in the precast units are stretched *before* the concrete is placed and the stress maintained in the wires until the concrete has hardened sufficiently to provide adequate strength. This can be done in one of several ways, some of which will be described subsequently. The joists can be erected without shores or intermediate supports of any kind, and the bottom flanges provide forms for the cast-in-place concrete. No steel setting is required on the site except for transverse rods and a comparatively light mat of temperature bars.

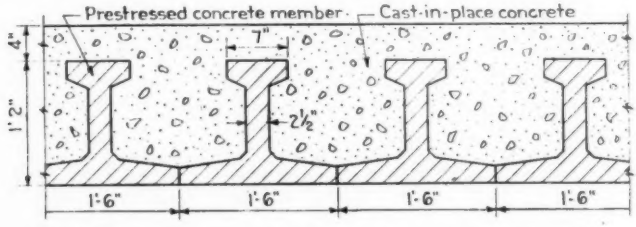

Fig. 3—Composite prestressed and cast-in-place concrete bridge deck

The joists are designed to support the load of the cast-in-place concrete and, in combination with it, to carry the live load. This type of combination construction has been used extensively in Europe. The prestressed units illustrated may be described as a modification of the self-contained units in which the Swiss firm Vobag specializes. This type of prestressed unit is self-contained in the sense that wires stretched in a factory remain prestressed by virtue of the action of the surrounding concrete which is precompressed. In the structure, the unit is placed where the prestressing is needed. The cast-in-place concrete is conventional reinforced concrete. It often may be economical not to prestress the entire concrete section but only the tensile zone.

LONG-LINE PROCESS

In 1943 the first factory was built in England for the production of prestressed railroad ties.[5] Wires for the ties in Fig. 4 are of 0.2 in. diameter. Long lengths of these are stretched and a steel form for a tie is placed around the wires on a vibrating table below a concrete hopper at the center of the production line. The concrete-filled form is conveyed by sliding it along the wires to within 4 in. of the form previously filled. Here the concrete is again vibrated and the surface troweled. One set of wires is used for many ties, and after the concrete has gained sufficient strength the wires are cut between the ties.

The railroad tie in Fig. 5 is made in France. Wire reinforcement is stretched separately for each tie and the protruding wire ends subsequently embedded in concrete.

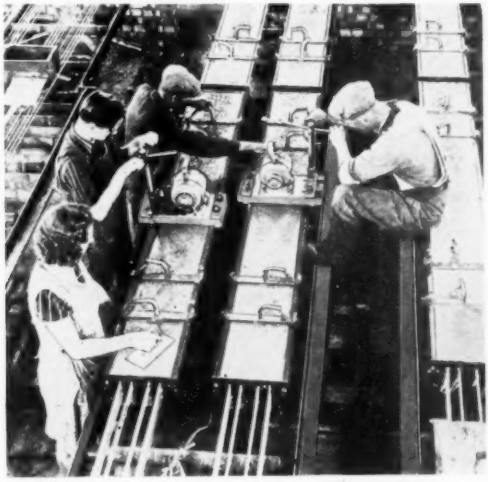

Fig. 4—Factory production of prestressed concrete railroad ties in England

Fig. 5—A French prestressed railroad tie

The London Midland and Scottish Railway used precast reinforced concrete for replacement of obsolete decks until in 1946 it began to experiment with prestressed concrete.[12] It was computed that the depth of the deck could thereby be reduced at least 15 percent which frequently would result in considerable economy, especially on replacement projects. The Adam Viaduct in Fig. 6 is the first such project completed. It has four spans approximately 30 ft long. The girders are I-shaped, 32 in. deep and placed side by side, six under each track and two under each railing. After the beams are placed, transverse tie rods are inserted in holes provided and grout is placed between the top flanges. The transverse rods are finally stressed to insure interaction of the beams.

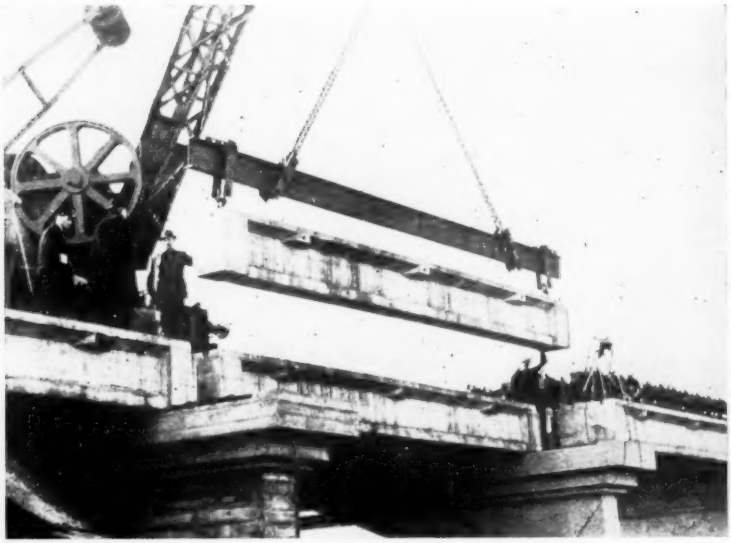

Fig. 6—Placing prestressed girder in Adam Viaduct

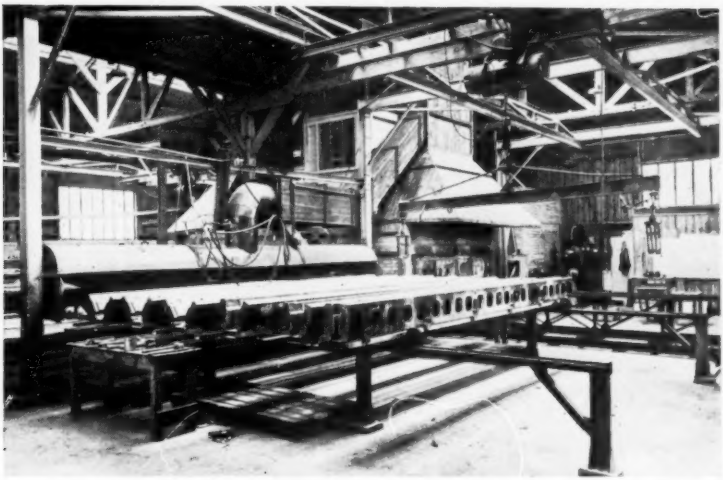

Fig. 7—Factory for producing prestressed concrete joists. Note hinged steel forms in foreground.

The prestressed units were made on a casting bed 930 ft long, and the wires were stretched by jacks placed at both ends. This length was chosen so that half of the units could be cast end to end, and only two sets of wires had to be prestressed for the whole job. Side forms were provided for eight beams and each set of forms was used eight times. After the concrete had attained sufficient strength, the jacks keeping the wires stretched were relaxed and the wires cut between adjacent girder ends.

Two of the beams were tested under 1.5 times the design load. Under that load no cracking was observed and the beams deflected only ½ in. or 1/720 of the span. No permanent deflection remained after the load was removed. The prestressed deck units were placed during three weekends, on one track at a time while traffic was maintained on the other. The average time for lifting and placing each beam was ten minutes.

INDIVIDUAL UNIT PROCESS

In 1945, a factory for making prestressed concrete joists was built in Orleans, France.[13] A general view of the factory is shown in Fig. 7. It is 70 ft square and produces about 800 ft of joists a day. The forms are made of steel and their sides are hinged to the base. They are first cleaned and sprayed with oil in the spray chamber. Wire reinforcing

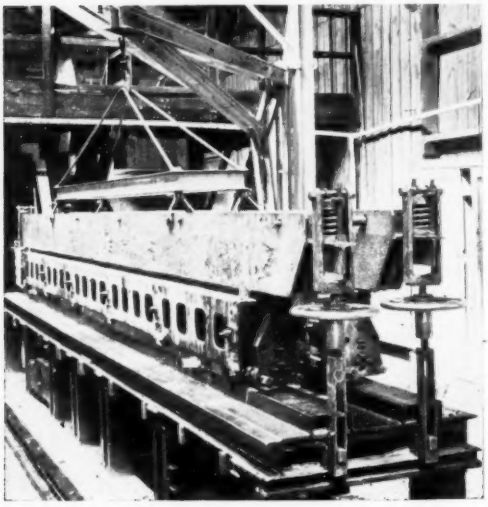

Fig. 8—Apparatus for vibrating concrete subjected to pressure

is then attached to yokes at the ends of the forms. One of these yokes, shown in Fig. 8, is movable. When the movable yoke with the wires attached has been extended the correct distance by a jack, its position is fixed by three set screws bearing against the end of the steel form. The jacking equipment is disconnected and immediately available for the next joist.

A pair of such forms are set on a vibrating table as shown. The concrete is placed, after which pressure is applied at the top of the concrete by means of a steel beam and heavy springs. The table is vibrated vigorously and under the combined action of pressure and vibration a considerable amount of water is squeezed out of the concrete. The joists are then steam-cured for two hours at atmospheric pressure, immediately after which the joists are taken out of the forms. The concrete at this stage is strong enough to resist the prestressing forces and to prevent slipping of the wires. The cube crushing strength after two hours is said to be about 4000 psi. The combination of vigorous vibration, high pressure and steam curing is employed in many European prestressed concrete plants to utilize space and equipment with a minimum of delay.

In practically all pre-tensioned work, the wires are anchored by bond. A wire diameter of 5 mm (0.20 in.) is said to be the maximum size with which satisfactory bond can be obtained. For longer spans and heavier loads, it may be necessary to use larger wires and employ post-tensioned

cables. The second half of the chart in Fig. 1 contains the outline for the post-tensioned procedures.

CIRCULAR POST-TENSIONED STRUCTURES

Circular structures such as tanks, domes and standpipes, may be prestressed by a special procedure used extensively in this country.[9] After a cylindrical concrete wall has been constructed, a special type of wire-winding machine is suspended from a track placed on top of the wall. The platform, carrying coils of high-tensile wire, propels itself around the outer surface of the wall. As the platform circles around the wall, wire is let out through a die on the platform. The size of the die opening is selected so that a predetermined force is required to pull the wire through. As the wire is wrapped around the wall it is automatically stressed to approximately 140,000 psi. The platform may be raised gradually to give the desired spacing of the wire. The maximum speed of the machine is 7 miles per hour, and the wire winding on an average 1,000,000 gal. tank can be completed in two days. The horizontal wires finally are covered with a thin coating of pneumatically applied mortar. The amount of reinforcement in such a prestressed tank wall is approximately 75 percent less than in a similar tank of conventional design.

For a water intake at Montreal, prestressed pipes were used which were made in accordance with the principles described for tanks.[14] Each pipe was cast with its axis vertical, and high-tensile wire was wound around the pipe with an initial tension of 150,000 psi. This stress was controlled by a die. The die assembly moved vertically while the pipe was rotated on a turntable.

FREYSSINET'S PROCEDURE

The Nunn's Bridge, near Boston, Lincolnshire, England, is the first prestressed cast-in-place concrete bridge in England and was completed in 1948.[15] It is a reconstruction of an existing old three-span arched brick structure. One single 74-ft span was desired to avoid obstructing the waterway. The deck depth had to be as shallow as practicable and the dead load of the deck as small as possible so that the existing abutments could be used with a minimum of strengthening. The main five girders, as seen in Fig. 9, were I-shaped and the webs only 4 in. thick. I-shaped sections weigh less and are more economical to prestress than rectangular sections. The main girders are completed in Fig. 9, the diaphragms are being built, but the roadway slab is not yet constructed.

The unstressed cables consisting of 12 wires each are placed almost like ordinary reinforcing bars. They are usually sheathed in tubes of

Fig. 9—Prestressed bridge girders in place

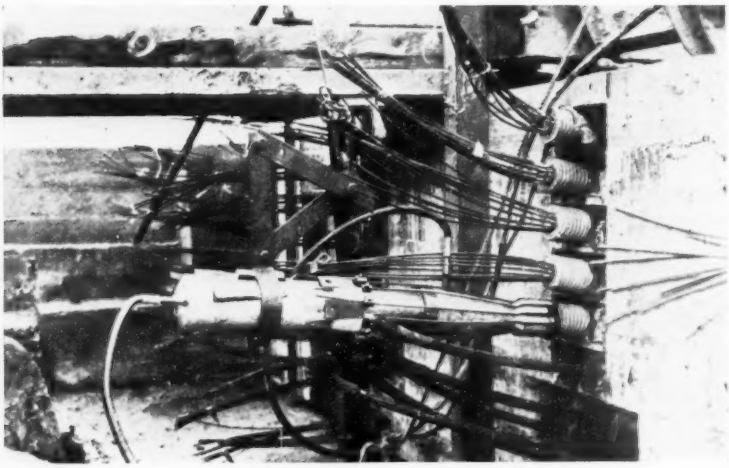

Fig. 10—Prestressing cables and Freyssinet jack

paper, metal or simply covered with bitumen to prevent bonding with the concrete. Some of the 12 cables in each girder are seen protruding from the ends of the girders in Fig. 10. The anchorage cones and the special double-acting hydraulic jack designed by Freyssinet are illustrated. The cable wires are spread apart at the ends and clamped to the jack by steel wedges. While one part of the jack induces and maintains the tension in the wires, another part of the jack forces a cone into the hollow space within the anchorage device and thereby clamps the wires. The wires were prestressed to 140,000 psi and the cables were made up on the job.

The Bourg d'Oissans Bridge in France[13] was completed in 1946. The wires were delivered in straight lengths and drawn through a bath of tar around a steel coil. They were then wrapped in paper and the cable tied with fine wire.

The girders weighed 55 tons each and were cast on a working platform, prestressed and moved laterally to their final position. The ends of the girders were precast. Some of the cables protruded through the top surface of the girders where a precast concrete unit was provided for anchorage of each cable end.

The concrete deck slab was cast after the prestressed girders were moved into place. The span is approximately 120 ft, the roadway 19 ft 6 in. and the girders about 6 ft deep, which corresponds to a depth-to-span ratio of 1:20. The wires had an ultimate strength of

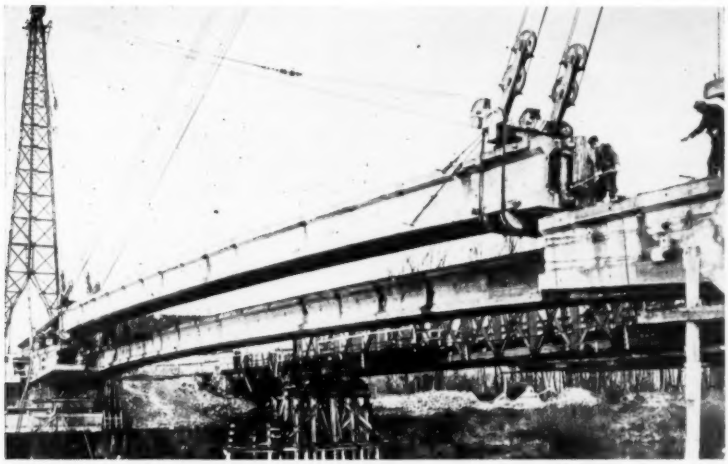

Fig. 11—Placing precast prestressed girder in Lucanzy Bridge in France

200,000 psi, and cubes kept at the site tested 4800 psi in 12 days. Two other bridges of the same type were built nearby at the same time.

The Lucanzy Bridge (Fig. 11), built across the river Marne, was completed in 1946.[16] This graceful structure is both in design and erection an outstanding example of skill and organization. Two existing shore abutments were 180 ft apart. The problem facing the engineers was to build a concrete bridge deck that could span 180 ft with a depth of only 4 ft 3 in. allowed between roadway level and navigation profile. The depth-to-span ratio is only 1:40. The deck in Fig. 11 comprises three hollow-box girders, each of which consists of 22 precast sections. Three of these sections were cantilevered from the shore. The 127-ft center portion of each girder consisted of 16 precast units assembled on the shore by means of temporary prestressing and then moved to the final position between the ends of the cantilevers.

The finished bridge is actually a shallow portal arch. The reaction is transferred from the deck to the abutment through an inclined concrete diaphragm. Special wedges at these diaphragms allow periodic adjustments in the horizontal thrust on the abutment. Several other similar bridges were subsequently built with precast units made in a factory.

The hangar at Karachi is one of two such structures recently built in Pakistan.[17] The roof consists of five barrel shells of concrete 2½ in. thick, which span a distance of 130 ft. The 9-in. wide edge beams were prestressed. Each beam had five cables with a total steel area of only 5 sq in., and the final prestress in the cables was 100,000 psi.

Over the front doors, the barrel shells and their beams are supported on a girder of 194-ft effective span (Fig. 12). This beam is I-shaped in cross section; the maximum depth is 22 ft; the flanges are 54 in. wide; and the web in the center is only 10 in. thick. This girder was also prestressed, and the total amount of steel in it was less than 10 tons. A steel girder of the same depth was estimated to weigh approximately

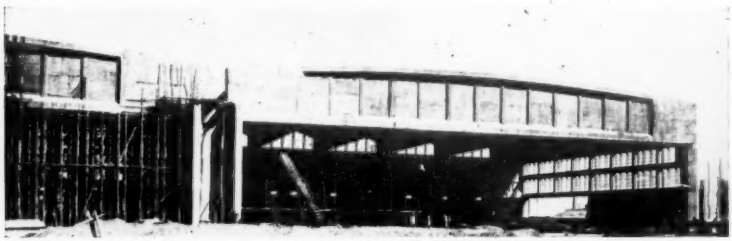

Fig. 12—Hangar in Karachi, Pakistan

100 tons. The work was so successful that it resulted in several other prestressed concrete hangars being placed under contract.

MAGNEL'S PROCEDURE

Magnel has developed and employed a cable and end anchorage different from those of Freyssinet. Fig. 13 illustrates the making of a typical Magnel cable. The wires are placed in layers of four, and as many layers are used as needed. The minimum clear distance between wires in both directions is $\frac{3}{16}$ in. Rubber inlays are placed in the girder in the exact position which is finally to be occupied by the cables. After the concrete is cast, the rubber inlays are pulled out and the cables inserted. After the concrete has reached its specified strength, the wires are stretched two at a time. Each pair of wires is anchored separately by a steel wedge and steel plates. The general arrangement of these so-called sandwich-plates is illustrated in Fig. 14. Steel bearing plates transmit forces from the sandwich-plates to the end surfaces of the concrete member. Finally, grout forced into each cable fully embeds the wires and establishes bond between the wires and the surrounding concrete.

At Brussels airport a group of four hangars has recently been completed.[18] The girders are hollow, cast on the ground as shown in Fig. 15, and finally jacked up in place. Their span is approximately 165 ft, their spacing is 33 ft and the maximum depth is 8 ft. Each girder contains 148 cu yd of concrete and weighs 300 tons. They were pre-

Fig. 13—Assembling a Magnel-type prestressing cable

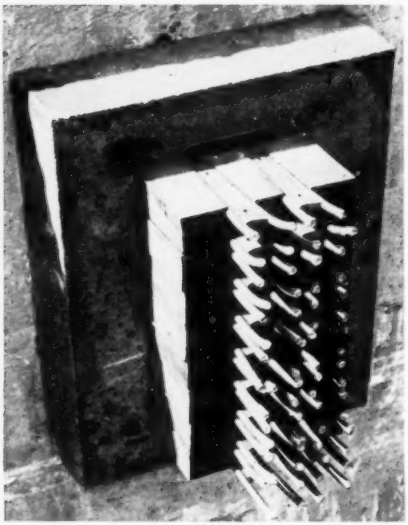

Fig. 14—Magnel end anchorage

stressed with cables placed inside the hollow cell and encased in concrete after the prestressing was completed. The consulting engineer states that "the contract was awarded in competition with steel and ordinary reinforced concrete. The prestressed concrete structure was not only economical but it required the least height, which for an airport is important."

The Sclayn Bridge[19] over the River Meuse in Belgium is probably the longest continuous prestressed bridge in the world (Fig. 16). The original bridge was wrecked but the piers and abutments could be used. The span lengths, therefore, were established beforehand. There were two spans of 206 ft each, which when made continuous made the prestressing cables more than 412 ft long. A separate 55-ft span was located at one

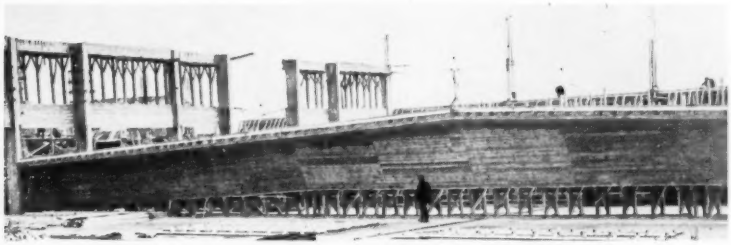

Fig. 15—Hollow prestressed beams 165 ft long cast on ground

Fig. 16—Sclayn Bridge under construction

end only. The authorities advertised for bids on two steel truss spans, each 206 ft long, but alternate bids were solicited. The prestressed concrete bridge was lower in cost than the steel trusses and was accepted. The deck was cast in place as a hollow-box girder with four 10-in. thick vertical walls and three intermediate cells. The top of this box-girder serves as the roadway. After the 36 cables placed in the cells were prestressed, they were encased in concrete for protection. This structure is one of the world's outstanding concrete bridges.

The textile factory in Ghent[18] is a one-story building covering an area of 375,000 sq ft or 8½ acres. It is an all-concrete structure and the roof is entirely precast (Fig. 17). The 100 girders and the 600 beams were all cast in steel forms on the ground, prestressed and then lifted to their respective positions in the roof. The cast-in-place columns were spaced 71 ft in one direction and 47 ft in the other. The girders were 5 ft 9 in. deep and provided with brackets to support the beams, which in turn support joists of 12-ft span. Finally, precast slabs were placed on both top and bottom flanges of the joists and the hollow

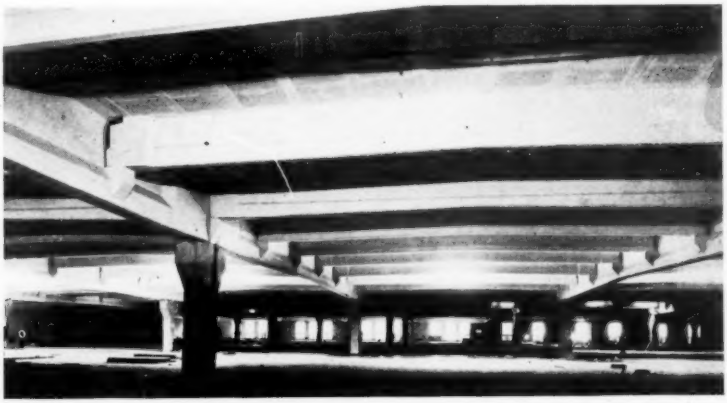

Fig. 17—Roof structure of textile factory. Note brackets supporting roof beams.

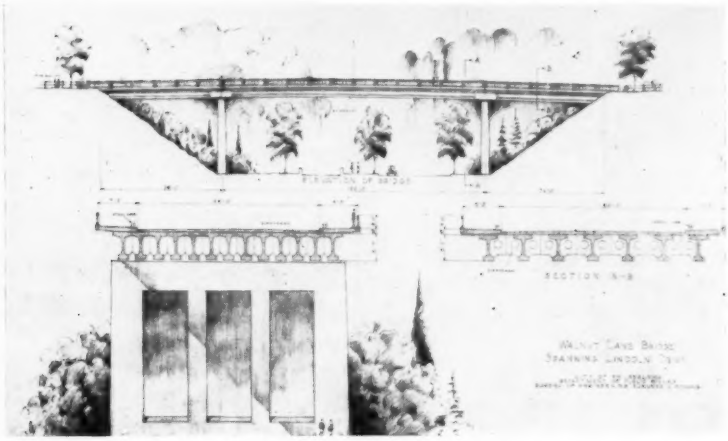

Fig. 18—Walnut Lane Bridge, Philadelphia

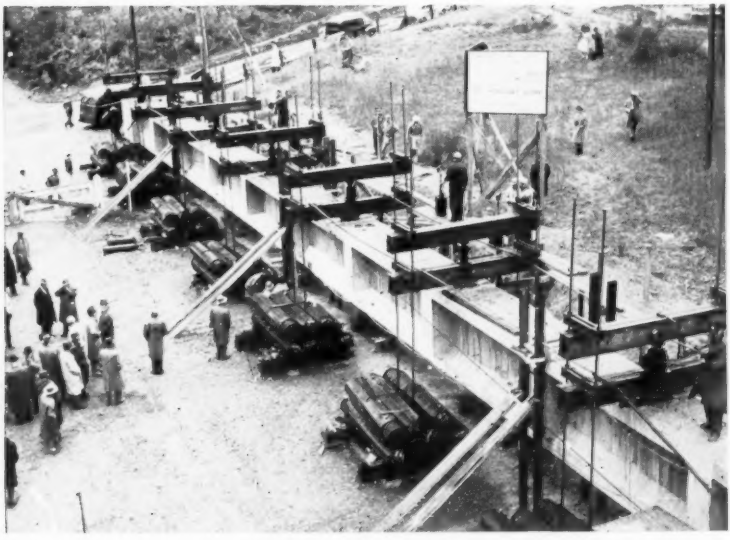

Fig. 19—Load test of Walnut Lane bridge girder

cells filled with glass-wool insulation. The designer, Professor Magnel, states that "this type of construction is very economical considering the wide spacing of the columns and the good insulation."

The Walnut Lane Bridge in Philadelphia, (Fig. 18) is the first prestressed structure of the kind in the United States. In 1948 a 150-ft span open-spandrel concrete arch bridge for this location was bid but not built because the cost was considered excessive. Other types of bridges were designed for this site but none proved acceptable to the art jury which must approve all structures built with City of Philadelphia funds. The prestressed bridge illustrated was then designed and is now under construction. All the girders are I-shaped, and the span lengths are 74, 160 and 74 ft. Girder depth in all spans is 6 ft 7 in. which gives a depth-to-span ratio of 1:24 for the long center span.

One girder, 160 ft long, was built and tested to destruction recently (Fig. 19). The test girder weighed 160 tons and contained 80 cu yd of concrete having a compressive strength of approximately 7000 psi at 28 days. It contained a total of 256 wires slightly more than ¼ in. in diameter. The wires were stretched 9 in. in pairs and anchored against the end surfaces of the girder. The total force needed to stretch all the wires was close to 1000 tons. Stretching the wires produced a maximum compressive stress of 2000 psi at the bottom of the section where tensile stresses exist in conventional reinforced concrete.

A loading capacity had been provided which made it possible to increase the load on the girder up to 10 times the working load, but this proved insufficient to cause failure. It was necessary to rearrange the ingots used for loading, and the girder finally failed under a load of approximately 15 times the working load for which it was designed. This was much more than anticipated.

CONCLUSION

All structural materials have their limitations in regard to span, load or height. Competition has a way of outlining areas in which each material and each structural type belong. Conventional reinforced concrete enjoys an enviable position and can be used successfully under most circumstances. But reinforced concrete has its limitations beyond which it encounters difficulties in competition. As spans become longer and buildings become taller, a point ultimately is reached at which weight of girders or size of columns begin to assert a confining influence. Prestressed concrete has one outstanding characteristic. It breaks down previous limitations on spans and loads, it opens up new fields beyond what is now considered concrete territory. It can compete with plate girders and steel trusses where reinforced concrete cannot. It is, in effect, a lighter and stronger material than reinforced concrete.

There is no reason whatever to believe that prestressed concrete will displace reinforced concrete. Prestressed structural concrete is an adjunct to reinforced concrete, something added to it but not essentially encroaching upon it.

Concrete with "built-in" compression is without doubt here to stay. It has so many pronounced advantages that it cannot help being a natural and logical step forward. Prestressed concrete is feasible, practical and competitive. The new type of structure can be built with the same cement, aggregate and concrete equipment as before. The main departure from conventional reinforced concrete is in the wire and in the methods of stretching and anchoring it. That is being accomplished now by numerous practical and economical procedures. The prominent fact is that concrete can be made crackless and free from tension. This step forward appears to be more promising for the future and to embody more potentialities than the original introduction of conventional reinforced concrete.

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Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

The problem of deleterious particles in aggregates

STANTON WALKER and DELMAR F. BLOEM, *Circular* No. 35, National Sand and Gravel Assn. 9 pp.

The nature and identification of deleterious particles are discussed. The effects of undesirable materials on the properties of concrete are given and methods of removing unwanted fractions are considered.

Bonded resistance wire gages for strain measurements

ARTHUR C. RUGE, *Production Engineering*, V. 20, Jan. 1949, pp. 116-117 APPLIED MECHANICS REVIEWS Mar. 1950 (Coffin)

Selection of the proper type bonded resistance wire strain gage is briefly discussed, consideration being given to the stress condition, gage length, gage size and gage resistance. Details of construction and principal dimensions are clearly pictured.

Economical design of a reinforced concrete fixed slab with straight haunches

YUKIMASA GOTO, *Journal of the Japan Society for Civil Engineers* (Tokyo), V. 35, No. 3, Mar. 1950, pp. 99-104

This paper reports a study of economical design of reinforced concrete slabs with straight haunches fixed at ends. The writer gives a diagram by which the most economical sections of a fixed slab with straight haunches can easily be designed without assuming the dead load of the slab itself.

Reinforced concrete skewed rigid-frame and arch bridges

MAURICE BARRON, *Proceedings*, A.S.C.E., V. 76, Separate No. 13, Apr. 1950, 31 pp. $0.50

This paper presents a method of analysis and design by which the effects of skew on a barrel arch or rigid-frame bridge may be evaluated separately. The skew problem is thus resolved into a sort of secondary stress analysis in which the ordinary stresses for a rectangular structure are used as primary stresses. The procedure is extended so that final steel areas of reinforcement and unit stresses are determined as functions of the areas and unit stresses for the rectangular structure. Equations and transformations are derived for all applied loads and for volume changes. Criteria are presented for the effects of skew. For single-span or double-span structures with less than critical skew, no skew analysis is required. For structures with greater than critical skew, an independent adjustment is shown for analysis and design.

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 21, No. 10, June 1950, *Proceedings*, V. 46. Address 18293 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

Making concrete

D. C. TEYCHENNE, *Building Industries and Scottish Architect*, V. 61, No. 719, Feb. 1950, pp. 49-53

Written for builders and contractors who mix concrete on the job, the importance of quality of materials, quality of mixing water, careful measurement of cement, sand and coarse aggregate, workmanship and curing are emphasized. Batching methods are compared and a field test for sand bulking is given.

Geometric method to determine the stresses in membranes of reinforced concrete

A. KLEINLOGEL and E. SCHMITT, *Bauplanung und Bautechnik* (Berlin), V. 3, Feb. 1949, pp. 51-58

APPLIED MECHANICS REVIEWS Apr. 1950 (Holl)

This is an exposition of a graphical method employed by Laponche, a French engineer, to determine the membrane stresses in cylindrical shells when bending stiffness is neglected. An example is given comparing the graphical method with known analytical methods.

Bibliography on machinery foundations; design, construction, vibration elimination

Bibliography No. 5, Engineering Societies Library, New York. $2.00

This is a list of 120 annotated references to selected books and periodical articles published from 1924 to 1949, on the theory, design and construction of machinery foundations. The references also cover specific problems such as heavy machinery foundations on unstable soils, and the vibration problems related to foundations of hammers, oil engines, electrical machinery, turbines, steam engines, compressors, machine tools, pumps and presses.

Concrete simply explained

VICTOR S. WIGMORE, Third and Revised Edition, 1950, The Society of Engineers (London) 48 pp. 2 shillings postpaid

Written primarily for the training of concrete workers, this booklet is written in simple nontechnical language readily understandable to anyone interested in producing good concrete. Familiarity with job conditions is reflected in the many work hints not ordinarily found in more technical treatises. The characteristics of cement, sand, coarse aggregates and water and their effects on concrete properties are considered with an additional chapter dealing with concrete practice.

Troublesome bubbles

GEORGE B. WALLACE, *Reclamation Era*, V. 36, No. 4, Apr. 1950, p. 66

Further information is given on the study reported in "Resistance of Concrete and Protective Coatings to Forces of Cavitation," ACI JOURNAL, Oct. 1949. Reproductions of high-speed motion pictures clearly show the implosion of cavitation bubbles under increased pressure. The introduction of 3 percent air into the flowing water reduced materially the damage to specimens tested in the cavitation machine. Other effective means of reducing cavitation damage were found to be proper design to avoid conditions favoring vapor bubble formation and the installation of resilient protective coatings.

Concrete pile making at Launceston

Constructional Review (Sidney), V. 22, No. 2, Feb. 1950, p. 28

The design of a plant for producing concrete piles 40 to 80 ft long was complicated by limited area and ground of low bearing strength. A pile foundation supports a

concrete working floor with a casting area for 60 piles and storage for a like number. Nominal 1:1½:3 concrete is mixed in a 20 cu ft tilting drum mixer, carried in bottom dump buckets by an overhead crane to the casting point, and placed in steel molds in which the reinforcement cages are positioned. Compared to average contract prices for similar piles a saving three times the initial cost of the plant was effected in the production of 2000 piles.

Apartment house unit (Unite d'habitation)

LE CORBUSIER, *Le Genie Civil*, V. CXXVI, No. 21, 1949 Reviewed by J. W. T. VAN ERP

The 20-story apartment house has as one of its many unusual features a reinforced concrete frame. The lower part of the building, for a height of about 3 floors, is an entirely open arcade, occupied only by the portals at 21 ft centers which support the upper structure. The total height is 180 ft which is little for the number of floors considering the great thickness of the floor construction required for extensive soundproofing, ductwork, and other utilities. At half-height the building contains shops, library, polyclinic, restaurant and a hotel. Except for the structural frame, prefabricated units of vibrated concrete are used extensively for walls, partitions, and floors. Apart from its construction features the project should prove interesting inasmuch as the owners expect it to be a paying proposition, its cost being considerably higher than the usual type of housing construction in France. The location, however, of its civic center at half-height for a tenant population of only 1600 seems to be of questionable merit.

Concrete curbs—Causes and prevention of failures

Road Research Technical Paper No. 18, Department of Scientific and Industrial Research, London. 18 pp. Available from British Information Services, New York. 25 cents

Although precast concrete curbs are widely used and have generally given satisfactory service, failures of isolated curbs or lengths of curbing are not unknown. This paper describes investigations carried out over the past three years in a co-operative research with the Cement and Concrete Assn. to find the causes of failure and suggest remedies.

In all the curbs studied the concrete was compacted by hand or by vibration; no failures of pressed curbs have been reported to the laboratory. The chief agency producing damage appeared to be frost; failure could not be related to the site conditions nor to the type of aggregate used.

The faulty curbs showed on the whole a higher absorption of water after 10 minutes' soaking than the sound curbs. Most of the failures were traced to easily remediable errors in manufacture such as faulty batching of the aggregates, too low a cement content, and, most commonly, too high a water-cement ratio. Practical suggestions for avoiding these errors are included in the paper.

New code for reinforced concrete construction in housing (Voorschriften voor gewapend betonvloeren in woningen)

Bouw, No. 11, Mar. 18, 1950, p. 184 Reviewed by J. W. T. VAN ERP

The existing concrete code in the Netherlands, the equivalent of the ACI Code, has been supplemented by an additional set of rules set up especially for the construction of housing; that is, lighter construction than that for which the existing code was intended. But under the impetus of the postwar need for economy and the trend towards an all out replacement of the scarce materials by the few plentiful materials, *i.e.*, concrete replacing wood where possible, a code was devised especially for these conditions, specifying mainly minimum sizes, minimum reinforcement and its location. These sizes are generally less than those required by the old code. For

example, floor slab thickness is only 2 in. instead of 3¼ in. At the same time allowable stresses are raised generally under the simultanuous requirement of strict supervision. This, however, as well as tests can be executed at the factory, thereby facilitating prefabrication.

The strength of thin concrete walls in axial compression under distributed loading

A. E. Seddon, *Final Report*, Third Congress, International Assn. for Bridge and Structural Engineering (Liège), 1948, pp. 589-596

Tests at the Building Research Station of the British Department of Scientific and Industrial Research to investigate the strength and performance of thin concrete walls are described. Data obtained from tests on 4-in. concrete walls of single-story height under distributed axial compression suggests that as the wall length is reduced from 6 ft 0 in. to 1 ft 6 in. the ratio between the ultimate mean compressive stress in the wall and the mean crushing strength of 4 in. cubes of the concrete used is reduced from about 0.75 to not less than 0.65. There was no evidence to suggest that the ratio is affected by the cube strength within a range of cube strengths from about 2500 to 5000 psi. The data were consistent with a fair average value of 0.75 of the ratio between the crushing strengths of 6 x 12 in. cylinders (tested between hard plywood packings) and 4 in. cubes, when the cylinders and cubes were stored together in air. In all the tests the wall failed by crushing of the concrete, and neither appreciable bending nor buckling occurred.

The thermal expansion of concrete

David G. R. Bonnell and Frank C. Harper, *Journal of the Institution of Civil Engineers* (London), V. 33, No. 4, 1950, pp. 320-330

Measurements of the coefficient of thermal expansion of different aggregates, and of concretes prepared from them with different cements, have been made over the temperature range 30 to 104 F.

The values for aggregates have been found to range from 2.1 to 6.5 x 10^{-6} per °F. Siliceous materials have the highest coefficients, limestones the lowest, and igneous materials intermediate. Concretes have values in the range 3.4 to 7.3 x 10^{-6} per °F. The magnitude of the coefficient of expansion of concrete depends largely on the aggregate used; concretes with siliceous aggregate have the highest values and those with limestone aggregate the lowest, while concretes with igneous aggregate have intermediate values.

The coefficient of thermal expansion of neat hydrated cement is higher than that of any aggregate measured. The richer mixes tend to have the higher thermal expansion but the order of difference is not high; the age of concrete has little effect on its thermal expansion.

The method of curing and the water content have only a small influence on the thermal expansion. Desiccated and water-saturated concretes have similar coefficients of expansion but the values are lower than that of partly-dry concrete.

Aggregates and their influence on durability of concrete

Kenneth B. Woods, *The Crushed Stone Journal*, V. 25, No. 1, Mar. 1950, pp. 21-26

Several references to studies of the performance of aggregates in portland cement concrete are cited to emphasize the importance of the characteristics of the coarse aggregate and the desirability of testing the durability of materials of unknown quality. Present soundness tests are then considered and found not entirely adequate to predict the behavior of aggregates under field conditions. Results of surveys of Indiana pavements showed a direct relationship between the frequency of popups and the source of the coarse aggregate. On the other hand, no correlation was found between

susceptibility to blow up and the source of fine aggregate, the time of year of construction and traffic conditions. Blowups, mapcracking and general disintegration were found to be much less important in pavements constructed on pervious sand or gravel than when built on impervious soils. From this it was deduced that moisture in the pavement was the immediate cause of failure and laboratory tests of vacuum-saturated aggregates incorporated into concrete cast into test beams were made which under test gave good correlation with the characteristics of known good and bad aggregates. In the later group were laminated dolomitic limestones, cherty limestones, high calcium limestones and certain siliceous river gravels.

It is concluded that the effect of freezing and thawing of concrete containing aggregates in a near-saturated condition is the primary factor in their lack of durability in Indiana concrete. Air entrainment, drying of aggregate before incorporating into the concrete and the use of base courses, subgrade drainage and summer construction are suggested as aids to the improvement of durability where the use of doubtful or inferior aggregates is deemed necessary.

The effect of shrinkage on the deformation of concrete under sustained load

FREDRIK VOGT, Trondheim, 1949, 40 pp. Reviewed by J. R. SHANK

A concise review of the theory of plastic flow in concrete as it is known to date is given as an introduction.

The body of the paper is a mathematical study of the interdependence of moisture and shrinkage of curing effects with plastic flow. Reference is made to the contention by the late G. A. Maney some years ago that those working on plastic flow were being led astray by shrinkage effects mistakenly taken for plastic flow. The author sees some value in the contention by Maney while not agreeing with the general contention.

The author follows mathematically the travel of shrinkage effects from the surface to the interior of a concrete specimen during sustained stress. The development involves five phases and the results are difficult to apply practically. He shows that measurements for plastic flow have generally been confused with the shrinkage effects even when the specimen is long in the direction of the sustained stress or strain. The shrinkage of an unloaded specimen does not serve perfectly to eliminate all of the shrinkage effects because of the interdependence of the shrinkage and plastic flow actions. It appears that the work of the paper might be continued in reverse to eliminate the shrinkage effects on plastic flow tests and make possible the determination of more accurate plastic flow properties.

The paper serves to indicate the considerable amount of distortion and stress coupled with plastic flow acting as a relieving agency that occurs in the early life of any concrete structure while the initial shrinkage of curing is taking place. More work along the lines of this paper might serve to develop cautions to be taken in the design of concrete shapes to avoid bad combinations of shrinkage, stress and plastic flow and greatly improve concrete and reinforced concrete design.

The effect of static and dynamic loads on a reinforced concrete tied-arch bridge

ALLAN J. OCKLESTON, *Journal of the Institution of Civil Engineers* (London), V. 32, Mar. 1949, pp. APPLIED MECHANICS REVIEWS Apr. 1950 (Quintal)

Although this investigation was made primarily to prepare the design of a bridge on the Umhlatuzi River in Zululand, the final results have a much wider range of application. Impact allowance had been made in the design, but it was found desirable to check this allowance by conducting tests on a model which was structurally similar to the proposed bridge. General methods of design were then applied to the model as built and results checked experimentally to study the validity of different assumptions.

In the mathematical analysis of any tied-arch bridge, the chief uncertainty being the interaction between the deck system and the arch rib, five methods of calculation were tried, starting with the assumption that the deck system and the hangers were of negligible stiffness and increasing the importance of that deck stiffness to the maximum possible. Comparison of the calculated deformations with the measured ones were then made for two types of loading. In the same manner, the natural frequency of vertical vibrations was obtained by purely analytical means and compared with the measured value. Lateral vibrations were measured experimentally only, and so was damping, both values being applied directly to the design of the structure. Forced vibrations such as may be caused by locomotives were also studied, and finally some tests were run on the prototype before the bridge was put into service, as a check on the model investigation.

A number of conclusions are drawn among which are: the deck system in a tied-arch bridge acts as a single member in a way similar to that of a stiffening girder in a suspension system; furthermore, the approximate method of analysis overestimates the rib moments for concentrated loads, and there is justification for employing in this method a larger value for the flexural rigidity than is customary. Finally, natural frequencies of vertical vibration and lateral vibration of the hangers can be estimated by approximate methods which are rapid and sufficiently accurate for most purposes, but although amplitudes can also be calculated, the method is too laborious for practical purposes.

Thermal drilling of stone masses (Le forage thermique des materiaux pierreux)

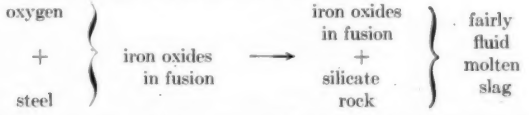
A. E. LEDUC, *La Technique des Travaux* (Belgium) V. 26, No. 3-4, Mar.-Apr., 1950

Reviewed by C. C. ZOLLMAN

This article describes in minute detail with sketches and photographs a method developed in Belgium for drilling holes in masonry masses. This method can be adapted to American practices and economy without any difficulties. The criteria which make thermal drilling practical can be summed up as follows: simplicity, flexibility, speed, economy and safety. The technique developed makes it possible to drill holes varying from 1 to 3 in. in diameter thru practically any depth of concrete, reinforced or not.

The shooting edge of a long steel tube of predetermined diameter is first heated with a blow-torch. As soon as this edge reaches a temperature required to initiate chemical reactions, oxygen is shot through the tube. Oxidation starts as steel brought to a certain temperature, called the initial temperature, burns in an atmosphere of oxygen while releasing intense heat. When the blow torch is removed the reaction continues, the generated intense heat releases fine drops of incandescent oxides which are now directed upon the rock. New reactions start as the incandescent oxides drops attack the silicate of the rock and form with it a fairly fluid molten slag. The slag is expulsed from the drilled hole by excess oxygen.

The entire process can be represented as follows:



This process can be used very successfully as practically all construction materials such as rock, stone and concrete contain silicate.

The article explains in detail the technique to be followed in the field, including drilling of holes in concrete under water, the little standard equipment required and how it is used. The detailed items such as quantities of tubes and oxygen used in terms

of diameter of holes are given and should enable the engineer to determine costs. This valuable article is completed with descriptions and photographs of actual applications.

Industrial structures in prestressed concrete (Voorgespannen beton in de utiliteitsbouw)
A. S. G. BRUGGELING, *Cement* (Amsterdam), V. 9-10, 1949, pp. 166-179 Reviewed by J. W. T. VAN ERP

The City of Orleans (France) is, like the City of New York, suffering from a water shortage. Unlike New York, however, this is due to a partial destruction of its brickwork storage tanks. To obtain new designs a competition was held under the following conditions: The footings for the new reservoir to be located above the old partly underground reservoirs, architecturally the designs had to conform to high standards due to its location within the urban area and the concrete walls were to be waterproofed by an inside layer of 1¼-in. high-test concrete with wire mesh reinforcement. Here again it was demonstrated that great savings were possible by prestressed construction. In conventional reinforced concrete construction 220 tons of steel were needed while the prestressed structure proved to need only 55 tons of mild steel plus 36 tons of high-strength steel, a saving of about 50 percent; the cement quantity was reduced by about 7 percent. Another advantage of the prestressed construction was the expected absence of tensile cracks so that the intended 1¼-in. waterproofing layer was replaced by a ⅝-in. coat of cement plaster.

The storage tanks were supported on columns set on the top of the old brick reservoirs. The new storage tanks themselves have post-tensioned beams, floors and walls. They are supported on the column heads by Freyssinet-type concrete hinges. These hinges are column sections, reduced to such an extent that the concrete, being in a plastic state (4500 psi), acts as a hinge. Floor slabs, continuous both ways, were for the first time, made in prestressed concrete. Although the structure was thoroughly designed and calculated, considerable difficulties were met during its construction, more to be expected in unusual prestressed structures than in conventional construction. Rectangular tanks in prestressed concrete have been built before but never as big as these which are 135 x 99 x 15 ft high. Nevertheless, they have fully proved their soundness in use and their economy in cost.

PERIODICALS REGULARLY SCANNED FOR REVIEWS

The many requests for information concerning the source material of reviews appearing in the "Current Reviews" section indicate the desirability of publishing a list of those periodicals being regularly scanned for articles dealing with the field of concrete. Included also are the publisher's addresses for convenience should correspondence with them be desired.

Betong	Brunkebergstorg 15, Stockholm, Sweden
Betong Og Jernbeton	15 Ridebanvang, Gentofle, Denmark
Beton-und Stahlbetonbau	Hohenzollerndamm, 169, Berlin-Wilmersdorf, Germany
Betonstein Zeitung	Bingerstrasse 14, Wiesbaden, Frankfurt Am Main, Germany
Bulletin, A.S.T.M.	1916 Race St., Philadelphia 3, Pa.
Bulletin, Institution of Civil Engineers in Ireland	35 Dawson St., Dublin, Ireland
Bulletin, University of Illinois	Urbana, Ill.

Cement	Sarphatistraat 54, Amsterdam C, Netherlands
Cement and Lime Manufacture	14 Dartmouth St., London (Westminster S.W.1), England
Cement och Betong	Malmo, Sweden
Cemento Portland	Calle San Martin 1137, Buenos Aires, Argentina
Civil Engineer Corps Bulletin	Bureau of Yards and Docks, U.S.N., Washington 25, D. C.
Civil Engineering	33 W. 39th St., New York 18, N. Y.
Commonwealth Engineer, The	349 Collins St., Melbourne, C. 1, Australia
Concrete	400 W. Madison St., Chicago 6, Ill.
Concrete and Constructional Engineering	14 Dartmouth St., Westminster S.W.1, England
Constructional Review	14 Spring St , Sidney, N.S.W., Australia
Crushed Stone Journal, The	1415 Elliot Place, N.W., Washington 7, D. C.
Current Road Problems	Highway Research Board, 2101 Constitution Ave., Washington 25, D. C
Giornale del Genio Civile	Via delle Vite, 13, Rome, Italy
Engineering Experiment Station News	Ohio State University, Columbus, Ohio
Engineering News-Record	330 W. 42nd St., New York 18, N. Y.
Hormigon Elastico	Lavalle 1171, 5°, Buenos Aires, Argentina
Handlingar (Proceedings)	Ingeniors Vetenskaps, Akademien, Stockholm, Sweden
Ingenioren	Copenhagen, Denmark
Journal of the American Ceramic Society	2525 N. High St., Columbus 2, Ohio
Journal of Research	National Bureau of Standards, Washington 25, D. C.
Magazine of Concrete Research	52 Grosvenor Gardens, London S.W.1, England
Osterreichische Bauzeitschrift	Mölkerbastei 5, Vienna 1, Austria
Polytechnisch Tydschrift	Haarlem, Netherlands
Public Roads	Bureau of Public Roads, U. S. Dept. of Commerce, Washington 25, D. C.
Reinforced Concrete Review, The	94-98 Petty France, London S.W.1, England
Revista de Obras Sanitarias de la Nation	Charcas 1840, Buenos Aires, Argentina
Revue des Materiaux	75 Rue Lafayette, Paris 9, France
Rock Products	309 W. Jackson Blvd., Chicago 6, Ill.
Structural Engineer, The	2 Bream Bldgs., London E.C.4, England
Teknisk Tidskrift	Kungsgatan 37, Stockholm C, Sweden
Teknisk-Ukeblad	Oslo, Norway
Travaux	6 Ave. Pierre 1st de Serbie, Paris 16, France
Zement-Kalk-Gips	Biebricher Strasse,74,Wiesbaden, Germany